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Proceedings of the American Society of Civil Engineers

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January, 1959

Papers

Journal of the
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RADAR FOR RAINFALL MEASUREMENTS AND STORM TRACKING[a]

Glenn E. Stout[1]

ABSTRACT

It was recognized during World War II that radar is an excellent instrument for obtaining data on the spatial distribution of precipitation. During the past ten years considerable research has been supported by the Department of Defense at various institutions to determine the application of radar in Meteorology. Efforts have been concentrated on the quantitative measurement of precipitation and on the use of radar for the detection and tracking of tornadoes, severe winds, and hailstorms. As a result of studies at the Massachusetts Institute of Technology, the Illinois State Water Survey, the U. S. Weather Bureau, and the Department of Defense, networks of radar are now being installed by various agencies interested in weather. In some cases radar sets are being procured by industry to improve short-range forecasts of precipitation and weather which are pertinent to their operations.

Illustrations show that radar occasionally detects the initiation of a new precipitation area one to six hours before it reaches a regular hourly reporting weather station. The areal extent of precipitation is fairly accurately displayed and the relative intensity of precipitation can be determined. Changes in the intensity and nature of the precipitation as well as in exact timing of the event are readily determined. Precise quantitative measurements for all types of rains have not been obtained to date. Despite this limitation, the river forecaster, hydrologist, and meteorologist can make use of radar to improve their skill in short-range forecasting of rainfall and river-stages especially during flash floods.

By observing severe thunderstorms on radar scopes that are associated with tornadoes, the meteorologist is learning to track existing storms and perfect his short-range forecast of areas for future warning. The loss of life during the 1957 tornado season was greatly reduced through use of radar to predict approaching severe storms.

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a. Presented at the meeting of the Hydraulics Division of the ASCE in Chicago on February 25, 1958.

1. Head, Meteorology Section, Illinois State Water Survey, Urbana, Ill.

INTRODUCTION

The existing national raingage network does not provide a reliable estimate of areal rainfall and often hourly or storm data are not available concurrent with the event to river forecasters, meteorologists, and other users of rainfall data. The raingage merely measures the rainfall in the immediate vicinity of the collector. Numerous investigators[1,2] have shown the inadequacy of point measurements. Figure 1 is an example from one of the Illinois State Water Survey's raingage networks which illustrates the variability of storm, monthly, and growing season rainfall for a 400-square mile area in central Illinois. Records from fifty recording raingages were used in drawing the detailed isohyetal patterns.

During World War II it was observed by military personnel that certain frequencies of radar are detectors of precipitation. Preliminary studies were initiated after the war at the Massachusetts Institute of Technology, through support of the U. S. Army Signal Research and Development Laboratory, to determine the capability of radar to detect meteorological parameters.

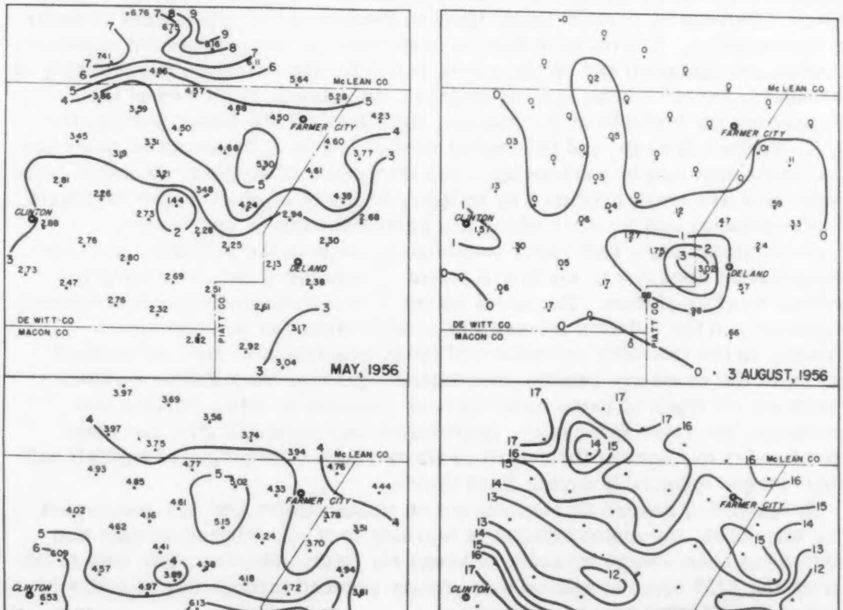

Figure 1. Daily, monthly and seasonal rainfall patterns for 400-square mile watershed.

In 1948 the Illinois State Water Survey became interested in the utility of radar for quantitative measurement of summertime precipitation in Illinois. Networks of closely spaced raingages were established to define the representativeness of rainfall observations and compare radar-measured rainfall with gage-measured point and areal rainfall.(3)

Numerous studies have been completed in the past eight years to determine the utility of radar in measuring rainfall. Preliminary findings of the various research organizations are presented in this paper in order to summarize progress to date. Difficulties have been experienced because research on quantitative precipitation measurement by radar have had to use discarded equipment of improper wavelength, electronically unstable and unsatisfactory test equipment. Despite several limitations, many applications of radar are apparent.

Principal of Radar

Radar is a high frequency radio device. It emits a short intense pulse of energy which is focused into a narrow beam of invisible energy by a rotating antenna, much like a search light. This pulse of energy travels at the speed of light. If the beam strikes an object, such as an airplane, a building, or a group of raindrops in a rain cloud, a small portion of the energy is reflected back as an "echo" to the point of transmission. The return signal is amplified and presented on a cathode ray tube, which is referred to as a Plan-Position-Indicator. The range and bearing of the object are automatically determined.

Research in this field by previous investigators(4,5) indicated that radar echoes from rain clouds are the result of back scattering of radio energy by raindrops falling through the atmosphere. The theoretical expression that has been developed indicates that the energy (power) received from raindrops may be expressed as follows:

$$P_r = K \frac{P_t}{r^2} \Sigma Nd^6 \tag{1}$$

where P_r is received power, P_t is power transmitted, r is the distance from the radar set to the reflecting raindrops, N is the number of raindrops per unit volume, d is the diameter of the raindrops, and K absorbs a constant for the refractive index of water and a number of constants which are parameters of the radar set.

Radar does not measure rainfall intensity directly, but measures the reflectivity, Z, which is proportional to ΣNd^6. Several expressions for rainfall rate in terms of ΣNd^6 have been obtained from raindrop size data. When one of these expressions,

$$\Sigma Nd^6 = K_2 R^{1.53} \tag{2}$$

where R is rainfall rate in inches per hour and K_2 is a constant of proportionality, is combined with equation (1), the resulting equation written in logarithmic form is, for the APS-15 radar system,

$$R = 1.4 \times 10^7 \left(\frac{P_r\ r^2}{P_t}\right)^{0.65} \tag{3}$$

Limitations

Before considering any of the research to date, it may be instructive to consider some of the assumptions that are made. The theory is based on scattering by spherical raindrops whose diameters are much smaller than the wavelength of the radar. The absorption of the signal, also known as attenuation, and the back-scattered energy are both increased by deviations from non-spherical drops. Most drops larger than 1.8 mm are non-spherical.

Another basic assumption is that the radar beam is filled with precipitating particles. Because the beam becomes larger as range increases, this assumption is not always correct for storms at long ranges, since the rainstorm may be smaller than the cross section of the beam. Also, since the height of the beam under normal refraction conditions is proportional to the square of the range, it is evident that at longer ranges the beam is higher in elevation and therefore less likely to be filled. At a range of 100 miles the height of the center of the beam is approximately 5000 feet. Owing to this limitation, present-day radar used for quantitative rainfall research should usually be restricted to 100 miles.

Another consideration of importance in quantitative precipitation measurements is that of the relationship of Z, radar reflectivity, to R, rainfall rate. Based on 1211 minutes of data, collected by the Illinois State Water Survey, using a raindrop camera,(6) Table 1 shows how R varies for given values of Z under different conditions of drop size.

TABLE 1

Z-R Relationships for Various Z

Z	R	R
	If $Z = 1119R^{1.52}$	If $Z = 149R^{1.62}$
mm^6/mm^3	mm/hr	mm/hr
10^3	.933	3.23
10^5	19.0	55.5
10^6	86.1	206

The table shows that the two Z-R relations yield rainfall rates that differ by a factor of 3 for the same measured Z.

Another consideration is that radar obtains a far greater sampling of raindrops in a shorter period than one can collect manually. In two years of operating the raindrop camera, Jones(6) sampled only 1211 cubic meters of raindrops, while one pulse from the radar at a distance of 30 miles samples 556 million cubic meters.

The fact that the radar is detecting a volume some distance aloft while the precipitation and drop size measurements are determined at the surface is a problem of indeterminate seriousness. Due to their smaller terminal velocity, small drops are carried farther horizontally than larger drops. This drift may become serious in a wind field where the vertical wind shear is appreciable. Considerable variation of reflectivity and rainfall rate in the vertical may occur under vertical wind shear conditions. Evaporation and coalescence of the drops may also change the reflectivity and rainfall rate between the radar volume and the ground. Thus, an error in surface rainfall estimation can be induced by those physical processes.

A further consideration is stability of the radar. Normally the receiver sensitivity is subject to fluctuation and should be monitored quite frequently.

Another rather serious limitation at 3 cm and shorter wavelengths is attenuation; that is, loss of signal strength due to intervening rain which absorbs a part of the transmitted and back-scattered energy. Robertson and King(7) determined experimentally that the attenuation is proportional to the product of rainfall intensity and distance into the storm. McGill University(8) and the Illinois State Water Survey using observed drop size data and Mie scattering theory found that attenuation per unit distance was related exponentially to the rainfall rate. Robertson and King's relation gives higher values of attenuation for rainfall rates below 90 mm/hr. Above 90 mm/hr the exponential relations give higher attenuation values.

Projects to Evaluate Radar

As part of a project at the Illinois State Water Survey, sponsored by the U. S. Army Signal Research and Development Laboratory to evaluate the usefulness of radar in estimating quantitative precipitation, areal rainfall amounts for one minute were compared with areal mean radar one-minute amounts. The rainfall amounts were obtained from a raingage network of 50 gages in a 100-square mile area located 20 miles northwest of the radar station. Radar-indicated amounts were obtained from a calibrated APS-15A, a 3-cm radar set. Areal means were determined by planimetering the radar isoecho contours.

Figure 2 shows the comparisons between radar power ratio and raingage amounts. The data(9) were derived from 95 minutes of summertime rains where no echo intervened between the radar site and the network. Cases of marked dissimilarity between radar and raingage patterns were also eliminated from the analysis. A lag up to 2 minutes between radar and raingage amounts was allowed to permit the precipitation to fall to the ground from beam height. Considerable scatter in the data is evident.

From recent analysis of an unusually heavy rainstorm,(10) the following conclusions regarding quantitative radar observations at 3 cm were presented;

"The CPS-9 radar set is satisfactory for portraying the areal extent of light rainfall, since attenuation effects are limited in light rain. In moderate to heavy rainstorms, this set does an excellent job of delineating the forward edge and the horizontal extent of an approaching storm in the absence of intervening echoes, as shown in Figure 3 for the October 9, 1954 storm over northern Illinois. At 1730 CST numerous rain cells are evidenced in areas between recording raingages. However, at 2030 CST neither the depth

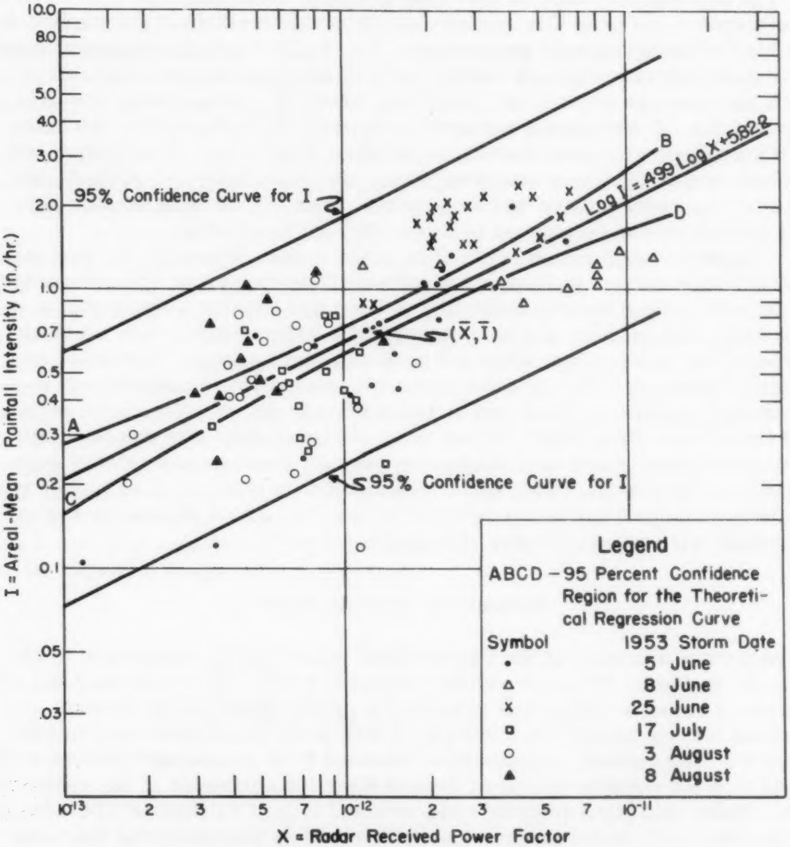

Figure 2. Relationship between rainfall intensity and radar-received power factor.

nor the intensity of a storm is accurately portrayed. Consequently, objective methods of obtaining accurate quantitative rainfall estimates with 3-cm radar do not appear feasible. However, an experienced radar meteorologist, using synoptic weather information and climatological relationships, along with knowledge of the limitation of 3-cm wavelength radar, is able to make better estimates of the rainfall in an area than can be obtained from analysis of the synoptic weather maps alone."

In another study the radar change in intensity was compared with the rain-gage intensity change. Using 5-minute amounts for cases of insignificant attenuation, it was found that 88 percent of the time the two parameters had the same sign change.

Using Robertson-King's equation, the Illinois State Water Survey has corrected a number of radar measurements of rainfall for attenuation. Cases where echo intervened between the radar were eliminated. Also cases of

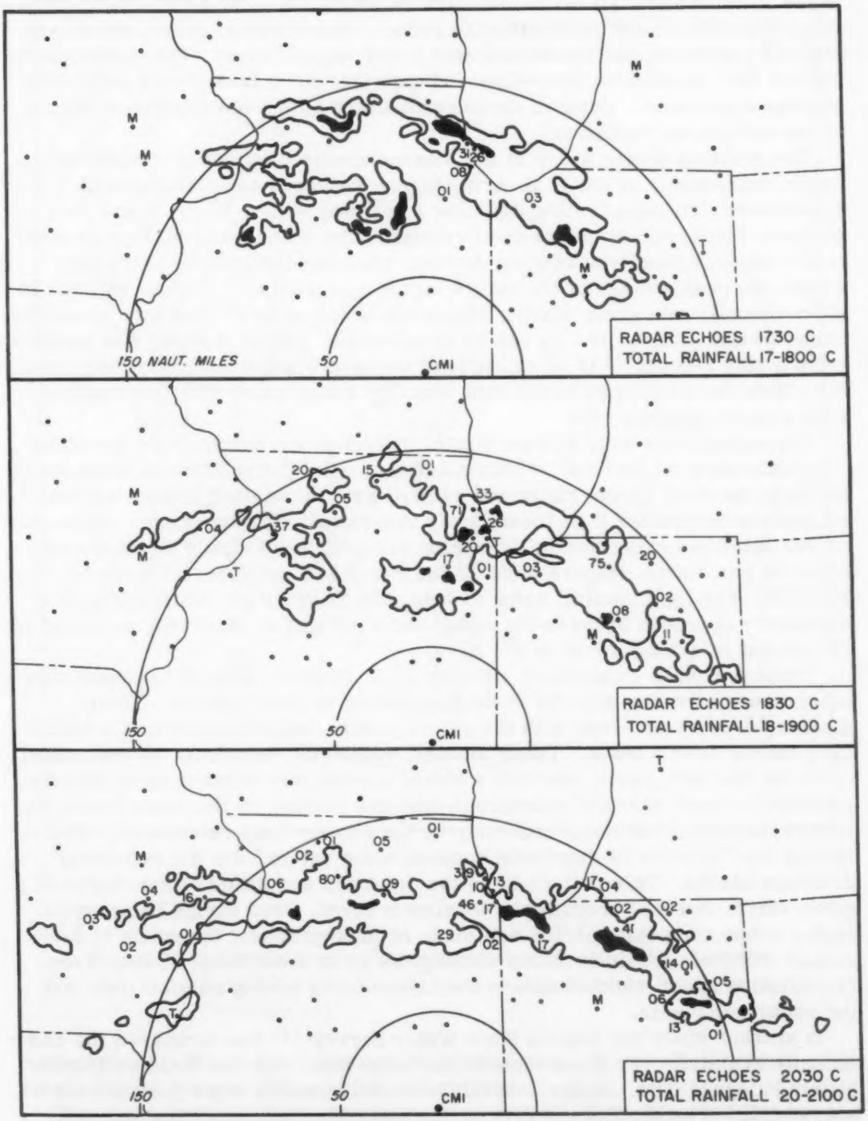

Figure 3. Radar echoes and hourly rainfall amounts for October 9, 1954 over northern Illinois and Indiana. (solid lines, full gain; solid areas, 26-DB reduction. All station rainfall values are zero unless otherwise indicated.)

strikingly dissimilar isohyetalisoecho patterns were eliminated. Results based on a comparison of point rainfall indicate no discernible improvement in the 3-cm radar's ability to measure rainfall. Areal mean rainfall comparisons were also made in an effort to reduce observational error, but no significant improvement in measurement error was obtained. The unsuccessful efforts with attenuation correction indicate that other factors are important besides attenuation. Some of these parameters have been discussed earlier in the section on limitations.

The Weather Radar group at the Massachusetts Institute of Technology, under sponsorship of the U. S. Army Signal Research and Development Laboratory, has been testing its pulse integrator with a 10-cm radar as a method of determining intensity of rainfall. The pulse integrator is an electronic signal intensity averaging device. Utilizing the ranging unit of the radar, the peak voltage of the return signal at a point or along an azimuth is electronically averaged. Gating allows the scope to be divided into numerous areas so that rainfall values can be determined. Figure 4 which was prepared from a MIT report(11) is an example of the type of information readily available. Note the similarity in the data when the radar areal rate is compared with a point raingage rate.

Correspondence with MIT personnel indicates that satisfactory accuracy (approximately 50 percent) is obtainable, provided the relation between signal intensity on their 10-cm radar and rainfall rate is obtained from a series of measurements rather than from purely theoretical considerations. Also, one or two sensitive raingages in the region being covered should be used as a check in any storm. Figure 5 shows the results of some recent work at MIT.(12) For light rainfall rates of less than 1/2 inch per hour, there is a similarity in traces between the radar and a raingage. However, an error of 25 percent is evident much of the time.

Texas A & M in cooperation with the U. S. Weather Bureau has been interested in use of radar data for flood forecasting in small basins. Their studies(13) are concerned with the velocity, size, and orientation of a storm as it moves over a basin. These studies, employing data from 10-cm radar, point out the well-known fact that a storm moving downstream along a basin produces a much sharper hydrograph than one moving up the same basin, although the actual volume of rain may be the same. They recommend determining the "velocity of maximum concentration vector" for the individual drainage basins. This is defined as the direction and speed of movement of a given storm that will produce the maximum crest, other things being equal. Radar scope composite photos are made by photographing the scope at 5-minute intervals for an hour and making one print from these 12 negatives. Quantitative radar-rainfall data are obtained using hourly rainfall data and the radar composite.

In another study the Illinois State Water Survey(14) has compared the radar-indicated rainfall over the extended time intervals, with the Weather Bureau hourly raingage data. Radar-indicated rainfall amounts were summed for 3 hours at 135 Weather Bureau precipitation stations in Illinois and Indiana. Radar isohyets for the 3-hour period were drawn to fit these data. Figure 6 shows the results of this analysis. The assignment of values to these radar isohyets, based on the 3-hour recorded precipitation amounts, was attempted but proved difficult and in many cases impossible. A radar isohyet would pass through too wide a range of raingage values, so that any calibration was

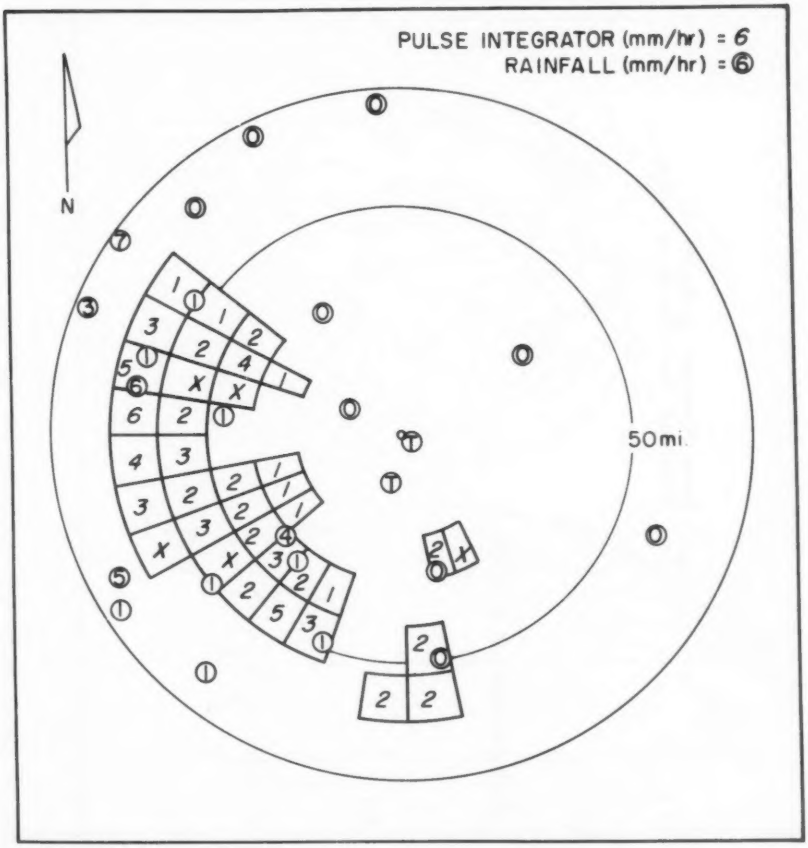

Figure 4. Comparative pulse integrator and rainfall values for July 10, 1952 (courtesy of Massachusetts Institute of Technology).

meaningless. The radar isohyets are labeled as A, B, and C. At times core displacement caused difficulty in calibration.

The University of Miami in cooperation with the U. S. Weather Bureau is comparing 2-hour composite photos with observed rainfall values, using an SP-IM, a 10-cm radar set. At 10-cm there is the advantage of less attenuation than at 3-cm. Since the radar scope, also known as the Plan-Position-Indicator (PPI), is an intensity modulated scope, the more intense areas over the period of integration will produce greater darkening of the film than less

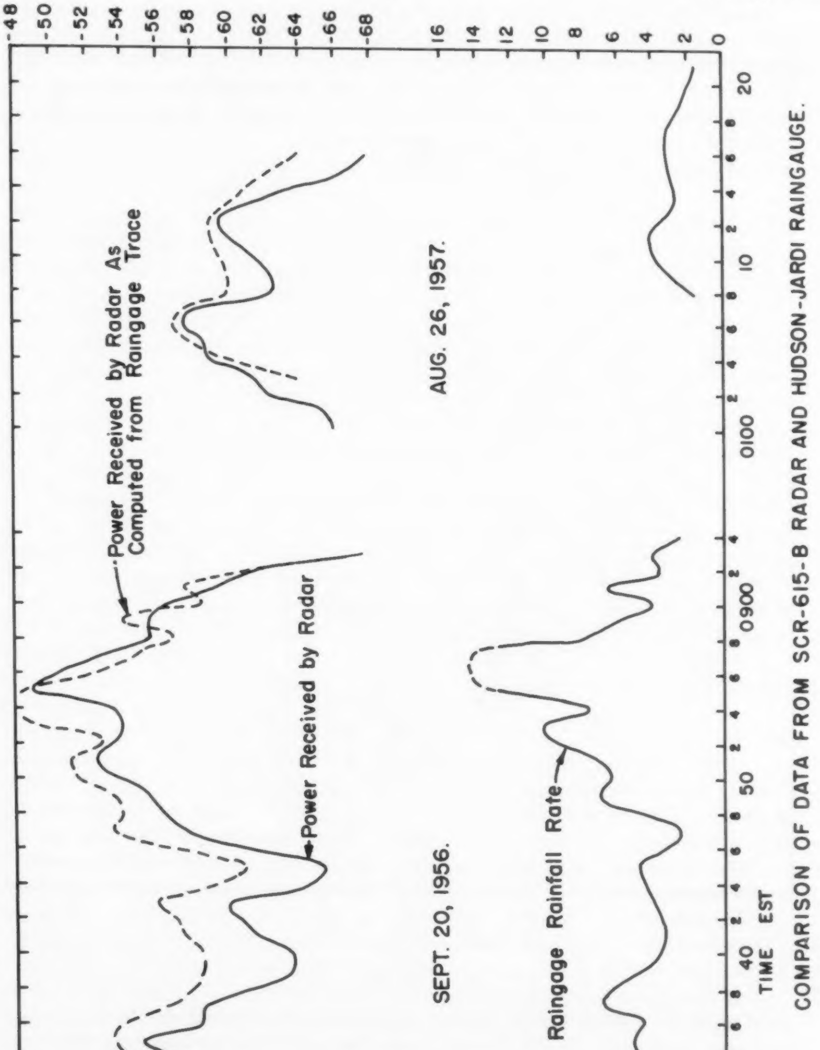

Figure 5. Comparison of data from SCR615-B radar and Hudson-Jardi raingage (courtesy of Massachusetts Institute of Technology).

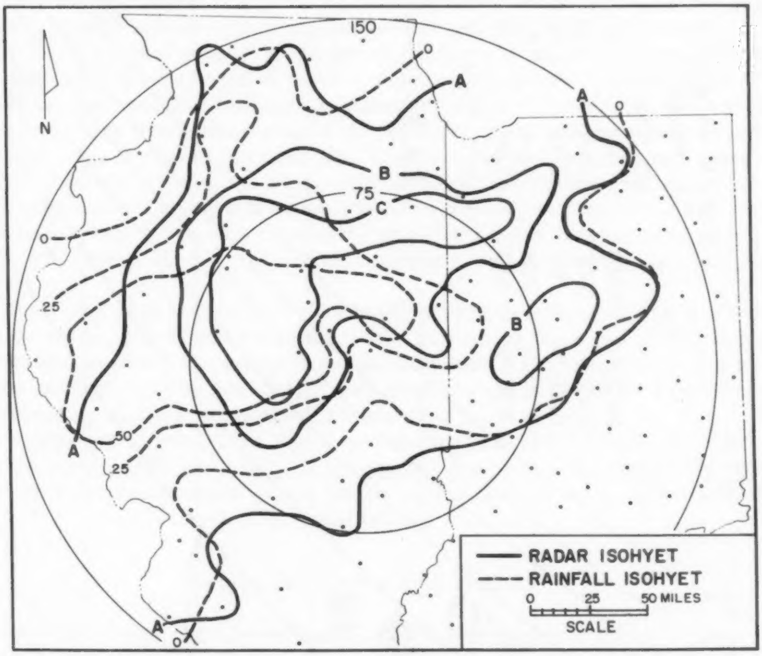

Figure 6. Comparison of 3-hour total rainfall and radar data for January 5, 1955.

intense areas. Also, a storm of longer duration will expose the film more than one of short duration. After many types had been tested, Kodak-Ektalure film was selected owing to its slow speed and wide latitude of exposure.

Many photographic and radar problems are involved in measuring rainfall quantitatively by radar. Critical control of the photographic developing process, film shrinkage, and halation are photographic problems. At close range, the scope becomes saturated. A logarithmic receiver instead of the conventional linear receiver is one solution to this problem. The fact that the velocity of the sweep in azimuth is proportional to range causes echoes at close range to appear brighter than those farther away. This effect is being compensated for by a filter, the density of which is inversely proportional to range. Furthermore, critical control of radar performance and setting must be maintained.

Figure 7 shows an opacity-range-rainfall nomogram prepared from these data.[15] Although the scatter on these diagrams is large, a potentially useful relation is indicated. This is a rapid and inexpensive way of supplementing hydrologic data.

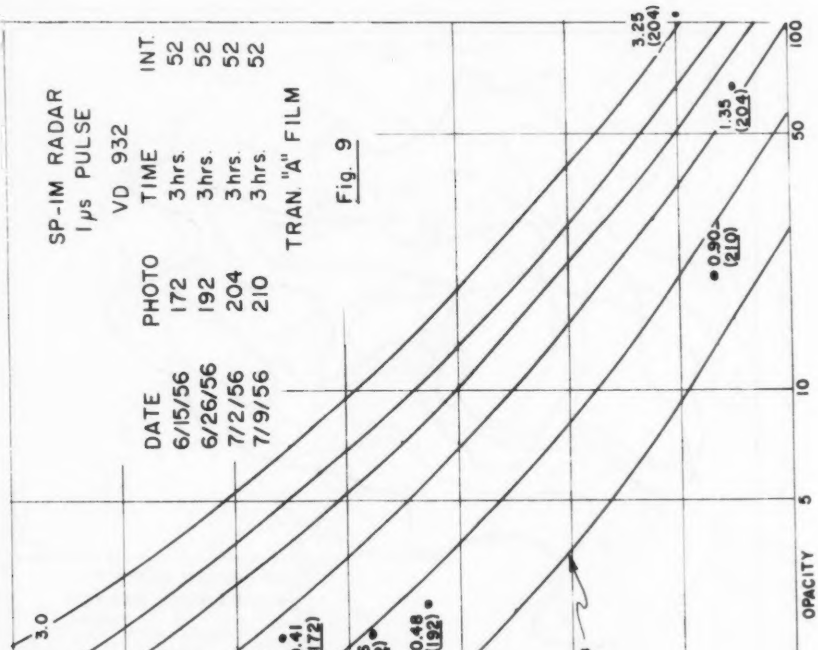

Figure 7. Comparison between opacity of film and range for various rainstorms (courtesy of the University of Florida).

Prospects

Future developments will probably include more use of photographic integration. This is a relatively inexpensive and quick way to obtain areal mean rainfall over several hours. More use is likely to be made of color presentation. Different colors could be used to represent a variety of intensities and, thus, enable an operator to judge the intensity distribution at a glance. Storage and memory tubes could enable the operator to survey the storm in time and space. Electronic computers may also be used but they are expensive.

Transmission of an actual scope picture to distant places may ultimately replace the coded message of today. Work is already in progress on transmitting the scope picture by facsimile, employing a low frequency to permit use of land lines. Thus, large scale composites can be made at an analysis center.

In the radar sets themselves we can expect the development of a greater dynamic range, which will eliminate scope saturation from the photographic integration process. Circuitry that will compensate for range attenuation may be developed. Constant height PPI presentation may add to a better understanding of the precipitation mechanism.

Development of equipment that would make radar information more accessible, useful, and reliable is in prospect. Analysis will most likely be aimed at determining areal precipitation over a several hour period and developing models of precipitation patterns.

Severe Storm Tracking by Radar

Since 1953 radar has proven itself to be a valuable instrument in detecting the areas of severe hailstorms and tornadoes. Incidental to Illinois State Water Survey studies of quantitative precipitation using radar, the first complete radar recording of the development of a tornado causing damage estimated at $3,000,000 was made.(16) Figure 8 illustrates the PPI for April 9, 1953, showing a hooked appendage developed on the southwest corner of a large thunderstorm. A tornado was centered under the vortex part of the appendage. Since then, numerous observations have been recorded by radars in Massachusetts, Oklahoma, Texas and Kansas. It appears that the maximum detectable range of hooked-shaped echoes is about 50 miles. Actually, the parent thunderstorm cloud may be detected at ranges up to 250 miles.

Exact data on the value of radar through better warnings during local severe storms is difficult to ascertain, since data on past storms are very incomplete. Experience has indicated that radar is very effective in the detection and tracking of severe storms. The network of radar stations needs to be expanded and improved upon. Better communications between the observer of the radar scope, the forecaster, and the public should be established. Eventually, the loss of life from tornadoes will be negligible.

CONCLUSIONS

Results to date on the use of radar to measure precipitation should be evaluated with respect to the user's requirements. Research studies indicate that point or areal measurement of precipitation by radar are feasible, but

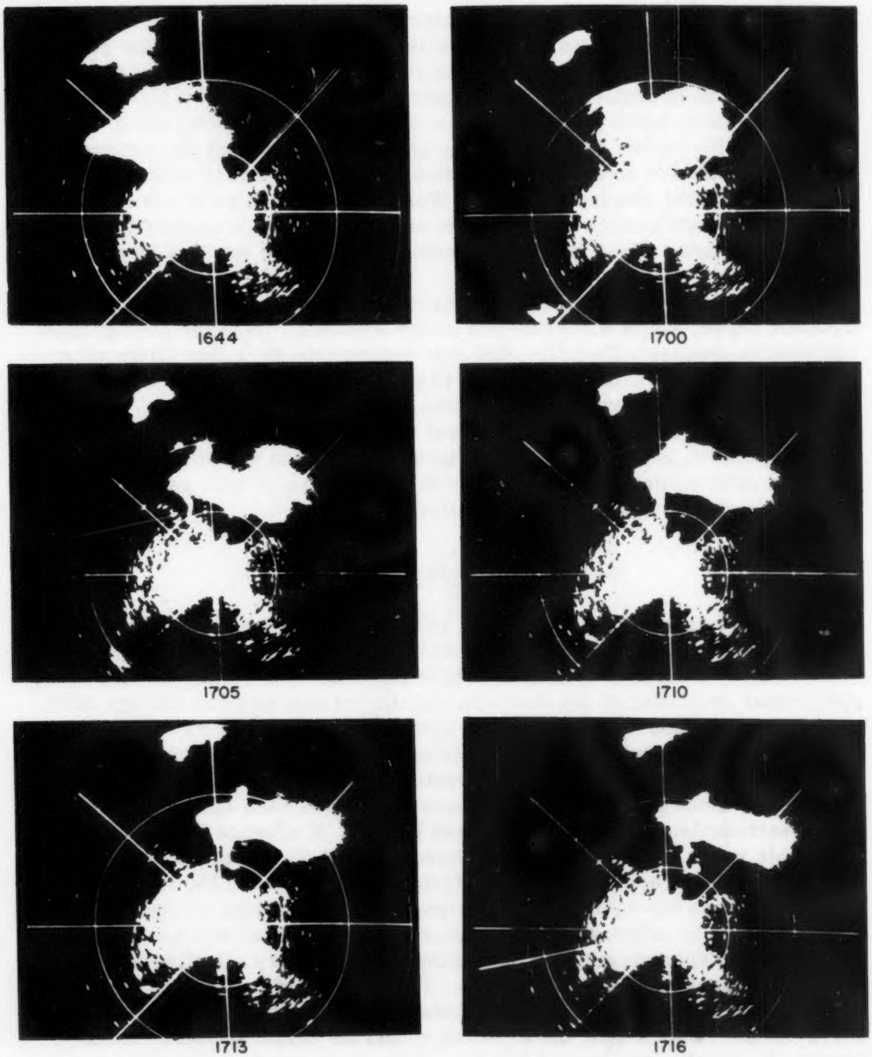

Figure 8.. Photos of radar PPI with 10-mile range markers showing development of cloud pattern associated with the tornado of April 9, 1953.

subject to discrepancies of considerable magnitude at times. The accuracy seems to be limited by the uncertainty in the empirical relation between signal intensity and rainfall rate which are based upon several assumptions. However, one must keep in mind that rainfall amounts from raingages are often not representative of areal mean rainfall.

The applications of radar in hydrometeorology are numerous. Concurrent with the event, the detection, areal extent, relative intensity and changes in intensity, duration, storm velocity, and orientation of precipitation with respect to the basin are readily recorded for the immediate use of the river or the synoptic forecaster.

Considerably more research and development of equipment should be pursued to solve some of the problems currently confronting the radar meteorologists.

Radar data will become more valuable as a network of stations is created, communication established to handle the enormous amount of data, and techniques developed to cope with the limitations of the data.

ACKNOWLEDGMENTS

The author appreciates the encouragement of William C. Ackermann, Chief, Illinois State Water Survey, to prepare and present this paper. Credit is due to the members of the Meteorology Section for performing much of the research reported on in this paper. Acknowledgment is also made of the other organizations for permission to reference their research.

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INTERIM CONSIDERATION OF THE COLUMBIA RIVER ENTRANCE

John B. Lockett,[1] M. ASCE

SYNOPSIS

To meet the increased needs of ocean navigation, the Federal project for improvement at the mouth of the Columbia River was modified by the River and Harbor Act of September 3, 1954, to provide for an entrance channel of suitable alignment having a depth of 48 feet at mean lower low water and a width of one-half mile. Under the modified project, this channel would be obtained by dredging and the construction of a spur jetty on the north shore, in combination with existing jetties and contraction works, at an additional cost currently estimated at $10, 340,000 for new work. The difficulties associated with prosecution of this modified project call for a re-evaluation of the natural forces controlling the regimen of the entrance area.

INTRODUCTION

The Corps of Engineers is attempting to secure by vigorous dredging a channel at the entrance to the Columbia River with least depth of 48 feet and of such width as may be permissible within the limits of available funds and dredging plant and the minimum requirements of present navigation. Although project depths of only 35 feet are authorized and maintained along the main Columbia River channel from the entrance to the mouth of Willamette River and thence to Portland, a distance of 117.5 miles, an additional usable depth of 13 feet through the entrance is necessary to assure safety to ocean vessels, particularly during the heavy sea swells and storms which occur at that locality. With this full depth assured, the physical capacity of the entrance to carry ocean commerce will be practically unlimited. However, maintenance

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1. Head, Special Projects Investigation Section, Planning and Reports Branch, Eng. Div., U. S. Army Engr. Div., North Pacific, Portland, Ore.

of previous project depths and the work of securing the new project dimensions have been made difficult, not only by adverse weather and sea conditions, but also over the years by the growth of the hazardous Clatsop Spit shoals. These shoals, lying along the channel side of the South Jetty, have gradually forced the navigation channel toward the North Jetty, as shown in Figs. 3 and 4, increasing annual dredging requirements and creating an undesirable alignment in the sea approach to the Columbia River entrance. Despite these difficulties, a usable channel, although less than full project dimensions, has been maintained. During the period 1946 to 1956, ocean traffic through the mouth of Columbia River increased from nearly 7 to over 12 million tons per year, as shown in the following tabulation:

YEAR	MILLIONS OF TONS
1946	6.9
1947	8.4
1948	7.4
1949	9.7
1950	9.2
1951	10.7
1952	10.9
1953	9.9
1954	9.3
1955	10.9
1956	12.6

History

A chart, made by Admiral Vancouver in 1792, clearly indicated the existence of a single entrance channel with depth of 4-1/2 fathoms between Cape Disappointment and Point Adams. In 1839, Sir Edward Belcher revealed the existence of two channels at the entrance separated by a large shoal area which forced the deeper channel northward against Cape Disappointment to turn abruptly southward and join a generally shallower channel along Point Adams. This shoal area, called the Middle Sands, grew in size until its 18-foot depth contour, in 1874, embraced a very large area seaward and south of Cape Disappointment, and covered almost all of the area now occupied by the present entrance. A portion of the Middle Sands formed an island inside the entrance which later separated from the main shoal area and, migrating to the north to form Sand Island, restored a single entrance channel to the mouth of the Columbia River in 1885. At that time, the main portion of the Middle Sands had moved westward to form a southward extension of Cape Disappointment and minimum depths of 20 feet prevailed on the ocean bar as shown in Fig. 1.

Pursuant to recommendations of the Board of Engineers for the Permanent Improvement of the Mouth of the Columbia River,(1) Congress on August 2, 1882, adopted the original permanent navigation project for the Columbia River entrance. This project provided for a bar channel 30 feet deep to be obtained by construction of a South Jetty, 4-1/2 miles in length, lying in a generally northwesterly direction from its base at Point Adams. Construction of the jetty was started in April 1885 and was completed, with four groins along its north side, in October 1895. At that time the crest of the structure sloped from 12 feet above mean lower low water at its base to plus 10 feet at a point 1-1/8 miles from the shore, tence to plus 4 feet at its outer end. A total of

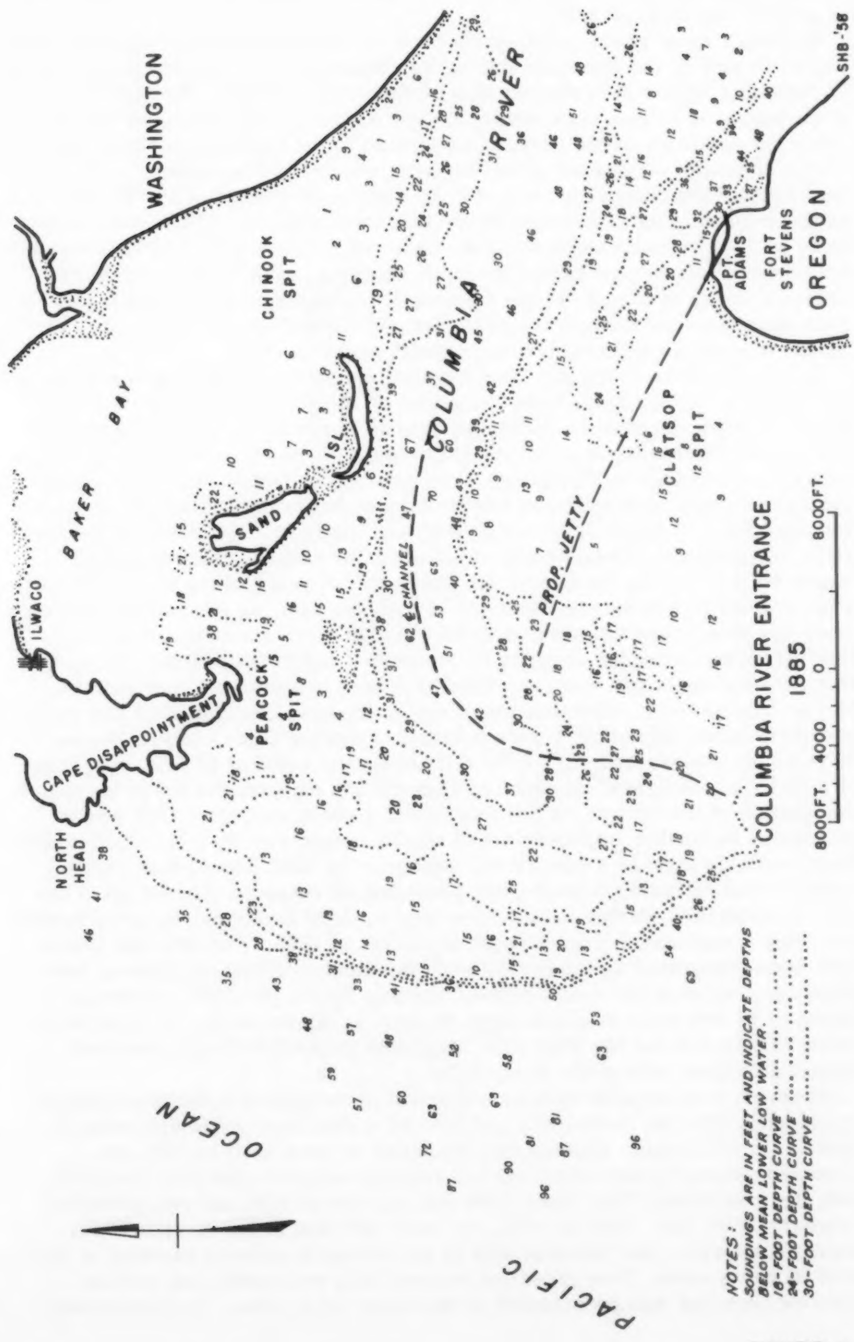
WASHINGTON
OREGON
BAKER BAY
ILWACO
CAPE DISAPPOINTMENT
NORTH HEAD
PEACOCK SPIT
SAND ISL
CHINOOK SPIT
COLUMBIA RIVER
CHANNEL
PROP. JETTY
CLATSOP SPIT
PT. ADAMS
FORT STEVENS
PACIFIC OCEAN
COLUMBIA RIVER ENTRANCE
1885
8000FT. 4000 0 8000FT.
SHB-'58
NOTES:
SOUNDINGS ARE IN FEET AND INDICATE DEPTHS
BELOW MEAN LOWER LOW WATER.
18-FOOT DEPTH CURVE
24-FOOT DEPTH CURVE
30-FOOT DEPTH CURVE

FIGURE 1

946,000 tons of rock was placed in the original construction of the South Jetty at a total cost of $1,969,000.

Although shoal areas marking the birth of Clatsop Spit began to form along the north side of the structure during its construction, a general improvement of navigable depths over the bar was evident in this period. By 1895 minimum depths of 31 feet were available over the bar on an alignment which, under the influence of the jetty, had migrated to the north about three miles. These favorable conditions, however, were short-lived because of continued migration of the channel to the north; by 1896 near closure of the 30-foot depth contour on the bar occurred; by 1898 depths of only 29 feet were available; by 1901 entrance depths had deteriorated further; and in 1902, instead of one, there were three separate entrance channels, each being about 22 feet deep, as shown in Fig. 2. In the meantime, Clatsop Spit shoals had formed a hook-shaped island through the South Jetty at its mid-length which later extended to form a solid land connection with Point Adams.

In view of these conditions, and the need for increased depths to provide a margin of safety for navigation, Congress, in the River and Harbor Act of March 3, 1905, adopted the recommendations of the Board of Engineers on Project for Improvement of Mouth of Columbia River.(2) These recommendations provided for modification of the existing project to secure an entrance channel one-half mile wide and 40 feet deep by extension of the South Jetty, construction of a North Jetty and groins, and dredging. Extension of the South Jetty, completed to a total length of 6.6 miles in August 1913, restored a single entrance along its former alignment to the mouth of the river. By 1911 a depth of 24 feet was again available across the bar. As completed, the South Jetty had an average top width of 25 feet and an elevation above mean sea level of 10 feet from the shore to the "knuckle" end of the original project, from whence the height was increased by stages to an elevation of plus 24 feet at its outer end. Construction of the North Jetty began in 1913 and by the next year, when only about 1 mile in length, entrance depths had increased to 30 feet. As constructed, the North Jetty had a top width of 25 feet, side slopes of 1 on 1-1/2, and crest elevation of 28 to 32 feet above mean lower low water. Completion of the jetty to its full length of 2.4 miles in August 1917 was accompanied by further improvement of entrance depths to 40 feet, which depths were available through a narrow bar channel. By 1920, the 40-foot depth contours had retreated considerably providing an entrance channel about one mile in width; and, during the next five years, these contours retreated farther providing a minimum entrance depth of 45 feet in 1925. The next two years have been recognized as the period when the most satisfactory channel conditions prevailed at the mouth of the Columbia River. By 1927, minimum depths of 47 feet were available over the bar, as shown in Fig. 3. It is interesting to note that for the first time, depths in excess of 40 feet prevailed along the channel side of the North Jetty.

However, concurrently with improvement of navigation conditions, shoals developed behind the North Jetty and formed a solid land mass extending almost to its full length. Clatsop Spit continued to grow until by 1931 its westward and northward extension had reduced entrance depths to about 43 feet. Rehabilitation of the South Jetty was started in 1931 and completed in February 1936; then, later in 1936, the outer 500 feet of the structure was impregnated with a hot asphaltic mix in an attempt to prevent raveling at this point by heavy seas. This expedient was not fully successful and a solid concrete terminal was constructed at the outer end in 1941. Further growth

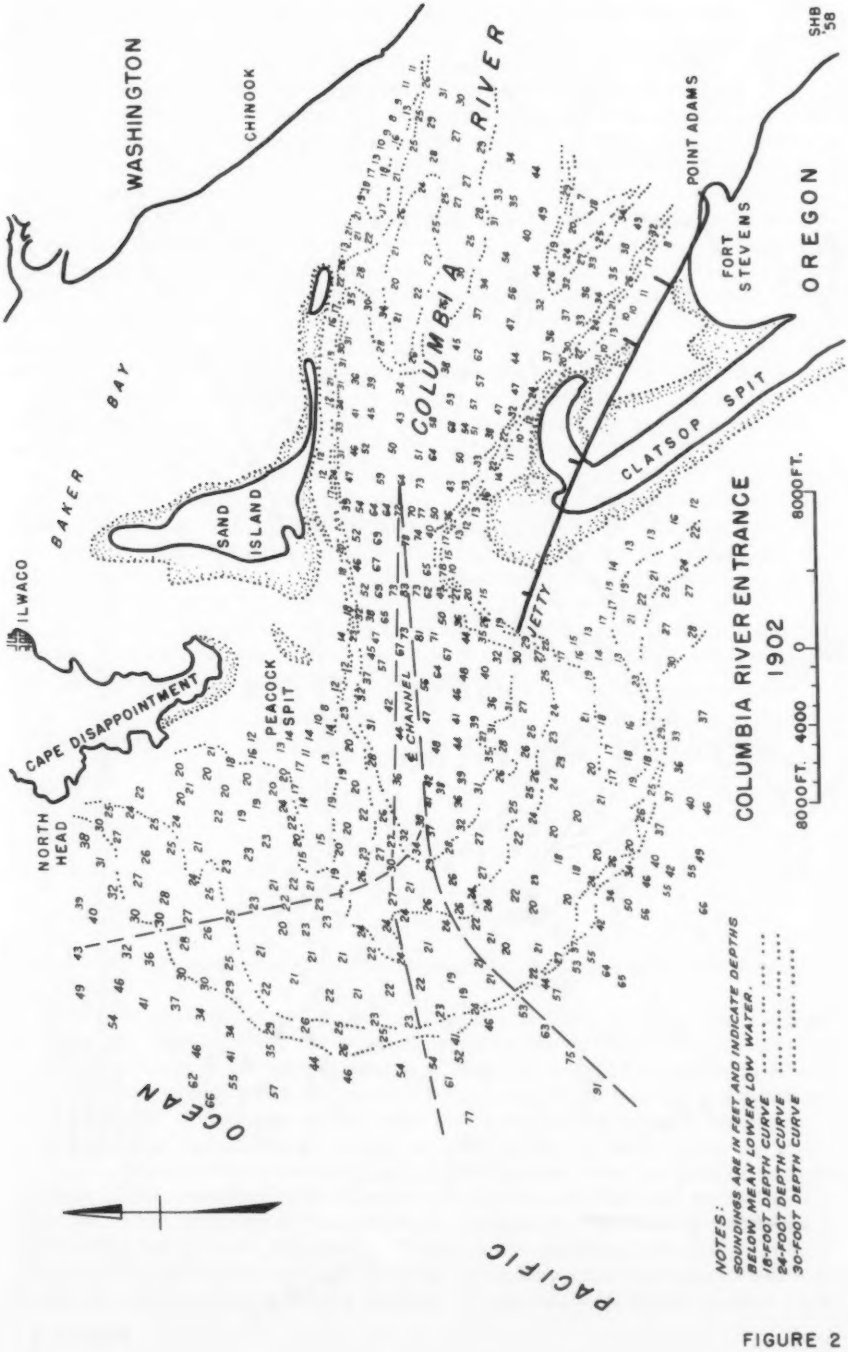
WASHINGTON
CHINOOK
COLUMBIA RIVER
POINT ADAMS
FORT STEVENS
OREGON
CLATSOP SPIT
JETTY
SAND ISLAND
BAKER BAY
ILWACO
CAPE DISAPPOINTMENT
PEACOCK SPIT
NORTH HEAD
CHANNEL
PACIFIC OCEAN
COLUMBIA RIVER ENTRANCE
1902
8000FT. 4000 0 8000FT.
NOTES:
SOUNDINGS ARE IN FEET AND INDICATE DEPTHS
BELOW MEAN LOWER LOW WATER.
18-FOOT DEPTH CURVE
24-FOOT DEPTH CURVE
30-FOOT DEPTH CURVE
SHB '58

FIGURE 2

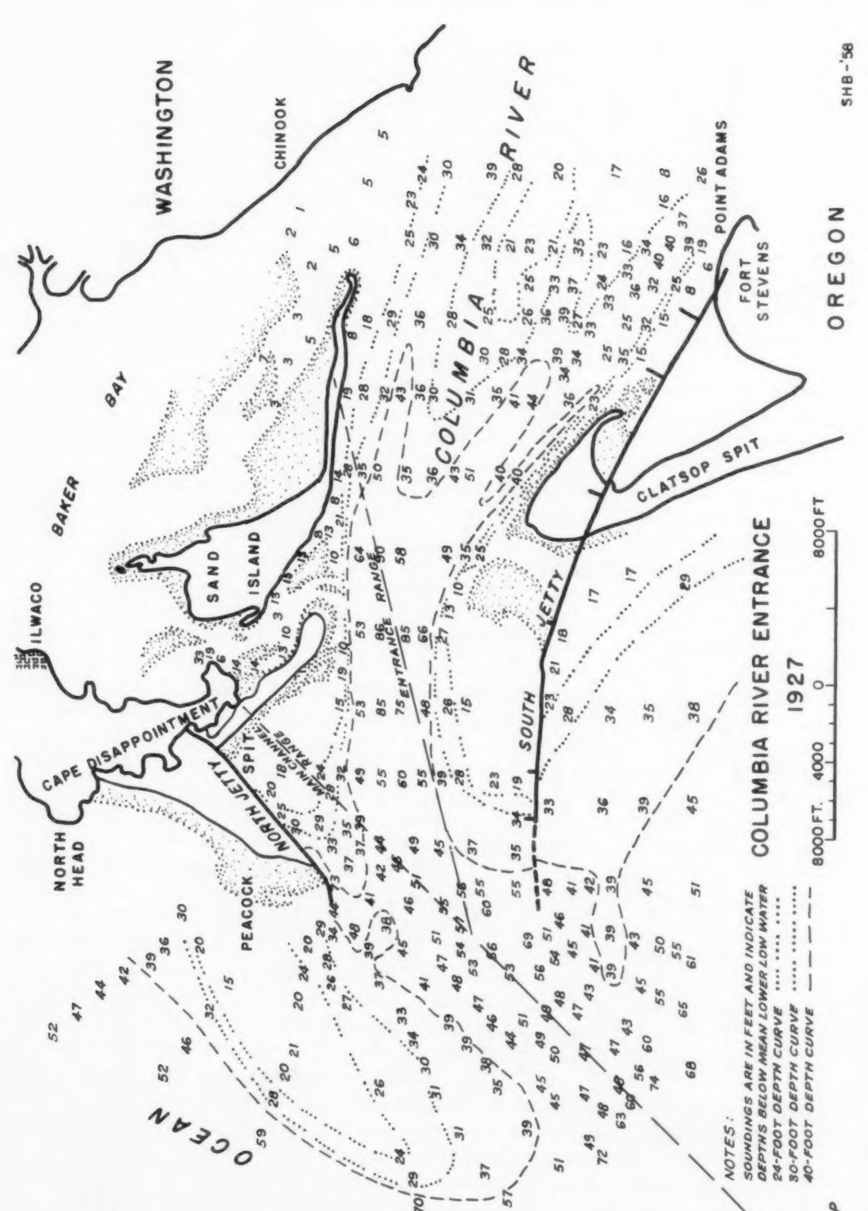
WASHINGTON
OREGON
CHINOOK
BAKER BAY
ILWACO
SAND ISLAND
CAPE DISAPPOINTMENT
NORTH HEAD
PEACOCK
NORTH JETTY
SPIT
MAIN CHANNEL RANGE
ENTRANCE RANGE
COLUMBIA RIVER
SOUTH JETTY
CLATSOP SPIT
FORT STEVENS
POINT ADAMS
PACIFIC OCEAN
LIGHTSHIP
COLUMBIA RIVER ENTRANCE
1927
8000 FT. 4000 0 8000 FT
NOTES:
SOUNDINGS ARE IN FEET AND INDICATE
DEPTHS BELOW MEAN LOWER LOW WATER
24-FOOT DEPTH CURVE
30-FOOT DEPTH CURVE
40-FOOT DEPTH CURVE
SHB-'58

FIGURE 3

of Clatsop Spit forced the navigation channel closer to the North Jetty placing deeper water along the channel side of the outer end of that structure. In an effort to prevent continued movement of the channel to the north, four permeable dikes were built along the south shore of Sand Island, and Jetty "A", extending southward from Cape Disappointment, was completed in 1939. The continued westward movement of Clatsop Spit resulted in the removal of more of the shoal area protecting the North Jetty thereby causing some damage from undercutting along the channel face of the structure. During 1938 and 1939 the jetty was rehabilitated and a concrete terminal block was placed at its outer end. By 1944, although the 40-foot depth contour opposite Jetty "A" had moved southward, this local improvement in conditions was accompanied by marked growth of Clatsop Spit to the west and north opposite the end of the North Jetty. Continued advance of the spit through 1951 coincided with definite movement of the 40-foot depth contour along the north side of the entrance channel to the east and south and in 1952, for the first time, an inner bar in extension of Clatsop Spit had formed between the outer end of the jetties.

In addition to 410,000 tons of stone used for rebuilding the outer end of the original South Jetty, a total of 4,427,000 tons of stone was placed in the extension of the South Jetty, completed in 1913, at a total cost of $5,657,000. Nearly 3 million tons of stone were used in the construction of the North Jetty at a cost of $4,319,000. Reconstruction of the South Jetty involved the placement of 2,288,000 tons of stone in the superstructure at a cost of $4,930,000. Rehabilitation of the North Jetty required the placement of 244,000 tons of stone and 234,000 tons were used in the construction of Jetty "A" at a total cost of $1,271,000.

As ample depths prevailed over the ocean bar, dredging under the 40-foot entrance project was confined to the face of Clatsop Spit. Amounts dredged since 1939 under that project were as follows:

FISCAL YEAR	AMOUNT DREDGED CUBIC YARDS	FISCAL YEAR	AMOUNT DREDGED CUBIC YARDS
1939	501,300	1949	1,023,800
1940	1,319,800	1950	927,200
1941-44	None	1951	1,000,300
1945	393,100	1952	1,267,400
1946	186,900	1953	2,796,800
1947	483,000	1954	2,141,700
1948	1,030,200	1955	1,749,300

The full project width was obtained only during the last three fiscal years listed above.

Under the provisions of the project adopted in 1954, the Corps of Engineers on April 15, 1956 initiated the work of dredging the 48-foot entrance channel; and, by the end of the dredging season in October, minimum depths of 48 feet over the full channel width of one-half mile through the inner and outer bars were obtained. These operations were accompanied by varied amounts of apparent scour and shoaling. Heavy shoaling of the channel during the non-dredging period between October and April, together with the lack of sufficient dredging plant, resulted in the decision to concentrate the 1957 dredging program to the securing of project depth throughout a channel of 1500-foot width along the adopted alignment. These later operations were also accompanied by natural scour and shoaling and heavy shoaling again occurred during the ensuing non-dredging season. Conditions prevailing in June 1957

are shown in Fig. 4. The following table shows the results of the 1956 and 1957 dredging programs and changes occurring within the adopted channel since initiation of the work as revealed by periodic condition surveys:

TABLE 1

DREDGING PROGRESS, MOUTH OF COLUMBIA RIVER, 48-FOOT PROJECT

DATE	QUANTITY OF MATERIAL IN AUTHORIZED CHANNEL, 1000 C.Y.	QUANTITY OF MATERIAL DREDGED DURING PERIOD 1000 C.Y.	APPARENT SCOUR OR SHOALING DURING PERIOD, 1000 C.Y.	
			SCOUR	SHOALING
Apr 15, 1956	18,911			
		1,301	1,428	-
May 1, 1956	16,182			
		1,554	-	2,746
May 15, 1956	17,374			
		1,371	1,158	-
Jun 1, 1956	14,845			
		1,392	-	1,396
Jun 15, 1956	14,849			
		1,424	-	274
Jul 2, 1956	13,699			
		1,467	2,242	-
Jul 19, 1956	9,990			
		746	1,403	-
Aug 1, 1956	7,841			
		1,841	32	-
Aug 26, 1956	5,968			
		1,495	-	696
Sep 15, 1956	5,169			
		1,186	-	95
Oct 5, 1956	4,078			
		659	-	3,285
Nov 7, 1956	6,704			
		0	-	1,067
Jan 14, 1957	7,771			
		0	-	1,728
Feb 4, 1957	9,499			
		0	-	2,301
Mar 4, 1957	11,800			
		0	257	-
Apr 6, 1957	11,543			
		373	1,459	-
Apr 29, 1957	9,711			
		766	1,590	-
Jun 3, 1957	7,355			
		723	-	390
Jul 10, 1957	7,022			
		755	-	1,219
Aug 7, 1957	7,486			
		1,142	1,229	-
Sep 18, 1957	5,115			
		150	-	606
Oct 10, 1957	5,571			
		0	-	1,309
Nov 5, 1957	6,880			
		0	-	1,635
Dec 11, 1957	8,515			
		0	-	941
Jan 21, 1958	9,456			
		0	-	1,355
Feb 19, 1958	10,811			
		0	1,575	-
Mar 13, 1958	9,236			

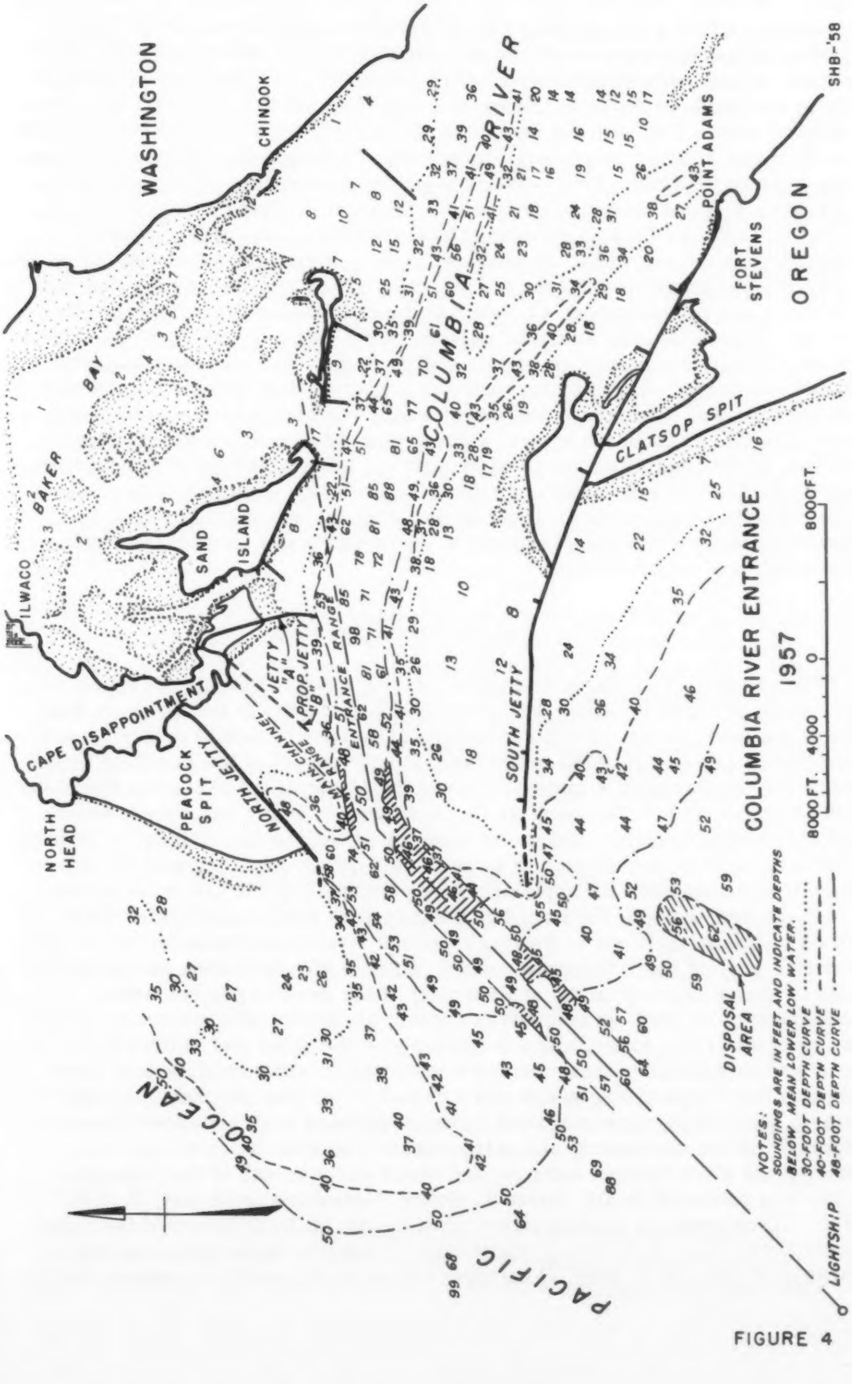
WASHINGTON
CHINOOK
OREGON
FORT STEVENS
POINT ADAMS
CLATSOP SPIT
COLUMBIA RIVER
SAND ISLAND
BAKER BAY
ILWACO
CAPE DISAPPOINTMENT
NORTH HEAD
PEACOCK SPIT
NORTH JETTY
SOUTH JETTY
JETTY "A"
PROP. JETTY "B"
MAIN CHANNEL
ENTRANCE RANGE
PACIFIC OCEAN
DISPOSAL AREA
LIGHTSHIP
COLUMBIA RIVER ENTRANCE
1957
8000 FT. 4000 0 8000 FT.
NOTES:
SOUNDINGS ARE IN FEET AND INDICATE DEPTHS BELOW MEAN LOWER LOW WATER.
30-FOOT DEPTH CURVE
40-FOOT DEPTH CURVE
48-FOOT DEPTH CURVE
SHB-'58

FIGURE 4

Statement of the Problem

During initial construction of the South Jetty, shoal areas, which later formed a land connection with Point Adams, gradually developed along the outside of the original structure. Almost concurrently, shoaling occurred along the inside of the jetty forcing the main channel to the north. This latter shoaling intensified with the extension of the South Jetty and with construction of the North Jetty so that a large land mass, Clatsop Spit, now lies inside and along the South Jetty. This land mass and its surrounding shallow depths present a definite hazard to navigation. During the period January 1947 to January 1952, the U. S. Coast Guard has reported a total of five vessels grounding in the entrance. In addition, owners of tankships carrying petroleum products reported that two of their vessels had cracked in crossing the entrance and presumably had touched bottom. Although depths greater than 48 feet are available in a natural channel inside the outer bar, just northwest of the Clatsop Spit shoals, as shown in Fig. 4, this channel is particularly hazardous during severe storms and has a greater deflection angle than the adopted channel that encroaches on the shallower depths of the spit. It has been possible to maintain project depths throughout usable widths during a portion of the time along the adopted entrance alignment by vigilant dredging when weather and sea conditions permitted, but such work has been costly. The problem resolves itself to one of securing the authorized project dimensions along a suitable alignment at costs which will be commensurate with the benefits of improvement.

Factors Relating to the Problem

The problem of securing dependable depths along a suitable alignment at the mouth of Columbia River has presented a challenge to the Corps of Engineers for many years. The complexities associated with this problem were recognized as early as 1882, when the minority report of the Board of Engineers for the Permanent Improvement of the Mouth of the Columbia River(1) indicated that "* * * The peculiar circumstance of heavy sea waves, strong and alternating currents, and the prevalence of sand in the vicinity, in unstable positions, introduce obstacles of so serious a nature that they may be said to place limitations upon the applications of principles at the Columbia River * * *". Later, in 1903, the Board of Engineers on Project for Improvement of Mouth of Columbia River in its report, which formed the basis for the 40-foot entrance project, gave thorough consideration to all phenomena known at that time to have a bearing on the achievement of the desired project depths. Subsequently, in 1932, an extensive program of current measurements across one principal river range, about 5 miles above the outer end of the jetties, in combination with other measurements including bottom sampling was undertaken by the Corps of Engineers and a report (3) on this program contains data on sand sizes, currents, tidal flows, suspended load, and other items. A report(4) on the mechanical and petrographic characteristics of material forming the river bottom, estuary, and beach at the mouth of the Columbia River was prepared by Dr. Edwin T. Hodge, Consulting Geologist, in April 1934. These reports; studies under taken by Dr. M. P. O'Brien;(5) the Tidal Model Laboratory at Berkeley, California;(6) salinity observations by the Corps of Engineers in 1936;(7) together with periodic condition surveys by the

Corps of Engineers and data concerning tides, winds, temperatures, and related phenomena observed by other Federal and State agencies, form the bases for present knowledge of forces controlling the regimen of the entrance area. From these data there has been extracted the following information pertaining to factors which may have a significant bearing on the problem:

River Stage and Discharge

The streamflow pattern of the lower Columbia River is influenced by spring snowmelt runoff from upland areas and, to a lesser degree, by rainfall during the winter months. The low-flow period generally occurs during the months of September through November and during this period river stages may infrequently be as low as zero, the adopted low water stage at Vancouver. At this location, winter rains from December through February usually result in gage readings of 4 to 5 feet. Beginning in March, melting snows gradually increase river stages to greater than 20 feet at Vancouver during the latter part of May and through the month of June. River flows then gradually decline to the low-flow period of September through November. Near the mouth, the extreme low river daily flow is estimated at 60,000 cfs, the average flow at about 255,000 cfs, the average spring freshet discharge at about 625,000 cfs, and the maximum discharge at about 1,300,000 cfs. Occasionally, winter freshets along the lower reach of the river exceed the average spring freshet. The average low water slope in the estuary is about 0.05 foot per mile.

Tides

Tides at the mouth of the Columbia River have the diurnal inequality typical of the Pacific Coast with a long runout from higher high to lower low water. The mean range of tide is 6.5 feet, the range from mean lower low water to mean higher high water is 8.5 feet, and extreme tides range from minus 2.6 feet to plus 11.6 feet mean lower low water. Tidal effect extends upriver to Bonneville Dam, 140 miles, during periods of low flow. The tidal prism has been estimated at 782 square-mile feet. The current survey of 1932 revealed the following characteristics of tides at the entrance:

TABLE 2

TIDAL CHARACTERISTICS, MOUTH OF COLUMBIA RIVER

MONTH 1932	AVERAGE RIVER DISCHARGE, 1000 CFS	AVERAGE DURATION OF TIDES IN HOURS		AVERAGE TIDAL FLOW IN 1000 CFS	
		FLOOD	EBB	FLOOD	EBB
April	315	5.43	6.90	1,037	1,345
May	556	4.95	7.55	971	1,481
September	125	5.81	6.44	1,257	1,306

Winds and Storms

Two weather seasons having wide variance in wind and sea conditions generally occur each year at the entrance. The season of severe storms usually extends from October to April and storm winds, beginning generally

in the south quadrant, gradually veer to the southwest and finally blow themselves out as they move around to the west or northwest. Storms are often continuous for several days and are accompanied by extremely heavy seas mostly from the southwest. During the summer season, the conditions of wind and sea are much less severe than those of the winter season. From May through September, winds from the north or northeast begin in the morning, increase in force during the day, and diminish at night. Occasionally, during the winter and spring, there are several days when an easterly wind predominates and the weather is cold.

Waves

Waves in excess of 40 feet in height have been reported by the Coast Guard at the mouth of the Columbia River. Observations of wave conditions between June 1934 and May 1935, as reported by Dr. M. P. O'Brien, indicate that the predominate wave direction is from the southwest. Weighted percentage of observed waves in each direction were:

DIRECTION	PERCENT	DIRECTION	PERCENT
North	5.9	South	17.5
Northeast	2.0	Southwest	28.1
East	3.6	West	24.0
Southeast	3.3	Northwest	15.6

Sediment

Although the Columbia River is not considered to be a sediment-bearing stream and is relatively clear for the greater portion of the year, it is known that some material is carried in suspension during freshets. Measurements below the mouth of Willamette River made in 1922 and at Vancouver in 1950 indicated 120 and 138 parts per million, respectively, in suspension during the average freshet. This, however, does not include the bed load of coarser material which, available evidence seems to indicate, travels in waves that range from 8 to 14 feet high near the mouth of Willamette River but decrease in height downstream and have crests from 300 to 800 feet apart. Tests on Eureka Bar, midway between the mouth of Willamette River and the sea, reveal that these waves may move from 30 to 50 feet in a single day.

Littoral Forces

The significance of the part littoral forces contribute to the problem may be evidenced by the tremendous build-up of material behind the North Jetty which began with initiation of construction in 1913 and kept pace with the jetty as it was pushed seaward. About 48 million cubic yards of material have been deposited in that area and, except for the part along the beach below high tide, is now stabilized by a natural growth of grass and weeds. The fact that there has been no similar build-up of materials behind the outer portions of the South Jetty appears to point to a predominate littoral drift to the south along the coast in the vicinity of the mouth of the Columbia River.

Currents

Although no instruments were specifically employed for the purpose of determining subsurface current directions, the program of current measurements

undertaken in 1932 indicated that mean ebb tide velocities were greater than those of the flood tides for all stages of the upper river, and during freshet stages were much stronger. The highest velocity observed was 12.28 feet per second on the ebb tide at one-tenth depth. These observations appeared to indicate that at high river stages, ebb bottom velocities were much higher than flood bottom velocities for all percentages of time. At intermediate river stage, bottom velocities were lower than at high stage with ebb bottom velocities perdominating for about half the measurements. At low river stage, flood bottom velocities were generally predominant. Considering the magnitude of ebb bottom velocities as compared to flood bottom velocities and their greater transporting power, it appeared evident at that time that the ebb flow was sufficient to maintain a channel between the jetties of at least 40 feet deep at mean lower low water and that no further contraction of the entrance appeared necessary for satisfactory channel maintenance. The following is a summary of mean velocities measured in 1932 opposite Clatsop Spit at strength of tides:

Time of Measurement	Average Velocity, Ft. per Sec.
At Low Stage (September)	
Ebbs (Seven tides)	4.85
Floods (Seven tides)	4.51
At Intermediate Stage (April)	
Ebbs (Eight tides)	5.06
Floods (Eight tides)	3.85
At High Stage (May)	
Ebbs (Seventeen tides)	5.58
Floods (Seventeen tides)	3.36

During August and September 1954, the U. S. Navy Hydrograph Office undertook a series of observations at the entrance including use of the Roberts Radio Current meter to measure current velocities and direction at depths near the surface, at mid-depth, and near the bottom. Measurements were taken at a total of seven stations, four located along the entrance channel at least one mile outside the jetties, one about two miles south of the outer end of the South Jetty, and two just inside the jetties. Although these measurements were made under difficulties and, accordingly, are incomplete, it is interesting to note that, while measurements at depths near the surface and at mid-depth indicated a general movement of water into the estuary on flood tides and out of the estuary on ebb tides, measurements at three and possibly four stations appeared to indicate at the lower levels a general movement of water into the estuary even on ebb tides.

In January 1958, further observations of current direction and velocity, salinity, and temperature were made by the U. S. Navy Hydrograph Office in and adjacent to the Columbia River entrance. Measurements made at a point about four miles upstream from the outer ends of the jetties during a river discharge of 175,000 cfs indicated strong density currents. Upstream velocities at the bottom ranging from 0.6 to 4.4 feet per second were observed during times of downstream and slack currents at the surface.

Salinity

Measurements by the Corps of Engineers(7) in 1936 of the specific gravity of surface, mid-depth, and bottom samples at 21 stations along the lower 22 miles of the Columbia River estuary indicated a definite difference in salinity between surface and bottom waters. During periods of flood tide on April 9 and 10, when the river flow was about 150,000 cfs, the maximum observed density of surface water was 1.018 as compared to 1.023 at mid-depth and 1.024 along the bottom. During periods of half to almost full tide on August 5 and 6, when the river flow was about 165,000 cfs, the maximum observed densities were 1.007, 1.020, and 1.023, respectively. These observations appeared to indicate that on these dates the upstream limits of salinity penetrations were about 15 and 11 miles above the outer ends of the jetties, respectively.

A number of salinity measurements were made by the Oregon State College and the Oregon Fish Commission(8) on July 25, 1955 along the navigation channel in lower estuary area with findings as shown in the following tabulation:

TABLE 3

SALINITY OBSERVATIONS, COLUMBIA RIVER ESTUARY, 1955

STA (MILES ABOVE MOUTH) AND TIME	SALINITY 0/00 (PARTS PER THOUSAND) AND DEPTH					
	SURFACE	15'	20'	25'	35'	45'	BOTTOM
STA 1 (2.6 MI)							
0930	1.9	--	--	4.8	--	8.6	25.5
1535	4.8	--	14.5	--	29.0	--	33.3
STA 2 (4.8 MI)							
1000	1.5	2.1	--	12.0	--		24.1
1600	5.5	20.8	--	--	35.8		32.5
STA 3 (6.8 MI)							
1030	0.8	1.2		5.0			24.3
1620	2.8	14.8		20.3			23.7
STA 4 (9.5 MI)							
1100	0.2	0.2		0.2	0.3		--
1625	0.0	0.8		6.6	--		14.8
STA 5 (12 MI)							
1125	0.0	0.0		0.0			0.0
1730	0.6	0.8		0.8			7.3
STA 6 (15 MI)							
1200	0.0	0.0		0.0			0.0
1755	0.4	0.2		0.0			0.0

Lower high tide occurred at 0528, higher low tide at 1138, and higher high tide at 1755 hours on July 25, 1955. River flow was 364,000 cfs on that date.

On the following day, these agencies conducted salinity observations at one station in mid-channel off Point Adams Coast Guard Station, Mile 7.5, with results as follows:

TABLE 4

SALINITY OBSERVATIONS NEAR POINT ADAMS COAST GUARD STATION, 1955

	TIME OF OBSERVATION AND SALINITY 0/00										
DEPTH	0700	0800	0930	1020	1215	1315	1415	1500	1616	1705	1745
SURFACE	1.2	0.9	0.0	0.0	0.4	0.6	0.3	1.1	1.7	2.4	3.4
5'	--	0.8	0.0	0.0	0.5	0.5	0.3	1.1	2.5	4.0	3.4
10'	1.5	1.2	0.0	0.0	0.6	0.4	0.3	1.8	4.0	4.4	4.0
15'	--	1.9	0.0	0.5	0.7	0.4	0.4	2.0	5.6	4.8	6.0
20'	9.8	3.2	0.2	1.1	0.8	0.4	2.9	2.1	5.9	5.8	8.1
25'	--	17.3	0.9	16.0	16.1	8.0	5.4	8.2	7.2	16.7	14.0
30'	17.7	19.1	11.8	17.7	17.0	11.5	7.0	11.0	8.0	17.0	15.2
35'	--	19.1	17.1	19.2	18.5	16.0	10.0	13.1	10.0	18.0	17.0
40'	--	19.0	18.4	20.1	20.6	19.6	16.2	13.1	14.0	18.4	19.3
BOTTOM	19.5	19.5	17.9	--	20.6	--	17.7	14.2	14.8	18.6	19.3

On that day, July 26, 1955, lower low tide occurred at 0041, lower high tide at 0626, higher low tide at 1223, and higher high tide at 1842 hours. River flow was 356,000 cfs.

Temperature

Observations of temperatures at different depths were made by the Oregon State College and the Oregon Fish Commission off Point Adams Coast Guard Station on July 26, 1955 concurrently with some of the salinity observations shown in Table 4 as indicated in the following tabulation:

TABLE 5

TEMPERATURE OBSERVATIONS NEAR POINT ADAMS COAST GUARD STATION, 1955

	TIME OF OBSERVATION & TEMPERATURE (CENT)				
DEPTH	0700	0800	1215	1415	1705
SURFACE	19.6	16.9	17.5	17.4	17.0
5'	16.0	15.9	17.5	17.5	16.9
10'	15.9	16.0	17.5	17.5	16.5
15'	--	16.0	17.6	17.3	16.3
20'	14.2	15.5	17.7	17.0	16.2
25'	--	13.0	15.0	16.4	14.9
30'	13.0	12.8	14.4	16.1	14.8
35'	--	12.8	14.0	15.8	14.7
40'	--	12.9	14.0	14.8	14.7
BOTTOM	12.6	12.9	14.0	14.8	14.7

Shoal Materials

Samples of bottom materials taken at 118 points in the entrance area during the 1932 current survey indicated that the coarser bottom deposits were found near the center of the section off Clatsop Spit and that seaward from this section the deposits became finer, the finest materials being found on the outer slope of the bar beyond the mouth. Bottom samples graded from coarse to fine from the navigation channel toward the jetties inside and from fine to coarse toward the jetties on the outside.

As the result of a study based on analysis of 199 samples taken in the vicinity of the entrance in 1933 and 1934, Dr. Edwin T. Hodge, Consulting Geologist, was of the opinion that the adjacent coast has in recent geological time been submerged and that the sea is awash against a cliff cut at a much earlier date. He believed that the materials derived from the erosion of the coast lie on the ocean floor as a submerged coastal plain and that the beaches are constructional, built from sand that has come from the upper reaches of the Columbia River and not from the adjacent cliffs. The clearness or freshness of the materials, their angularity, and the presence of several particles adhering together were considered as proof of the transportation of the grains by floating during a recent period of time. He concluded that the beach sands are not derived from the erosion of bedrock adjacent to the coast but originate from a distant source. As the source was considered one of about one-fourth andesite and two-thirds granite and rhyolite rock, it was believed that the Cascade Gorge could supply the former, but only the upper reaches of the Columbia River could supply the latter materials.

Sieve analyses of bin samples of materials dredged at the entrance within recent years show the following characteristics:

CLATSOP SPIT BIN SAMPLES

DATE OF SAMPLING	D_{10} GRAIN SIZE, MM	UNIFORMITY COEFFICIENT, $\frac{D_{60}}{D_{10}}$
July 23, 1952	.20	1.40
May 15, 1953	.20	1.30
June 8, 1953	.17	1.29
June 10, 1954	.23	1.30
July 26, 1955	.21	1.43
May 31, 1956	.18	1.55
August 22, 1956	.17	1.18
August 22, 1956	.13	2.00
September 16, 1957	.12	1.42

OUTER BAR BIN SAMPLES

DATE OF SAMPLING	D_{10} GRAIN SIZE, MM	UNIFORMITY COEFFICIENT, $\frac{D_{60}}{D_{10}}$
July 26, 1955	.12	1.50
July 26, 1955	.14	1.64
May 31, 1956	.12	1.83
August 22, 1956	.12	2.08
August 22, 1956	.15	1.60
September 16, 1957	.13	1.53

Near the end of the 1956 dreding season an area of hard packed clay, approximately 3000 feet long and 600 feet wide, was encountered at the 48-foot level along the centerline of the channel near the turn in alignment.

Channel Alignment in Estuary

Referring to Fig. 5, the adopted alignment of the navigation channel downstream from Harrington Point makes a long curve to the south past Tongue Point and Astoria, thence through a sharp reverse curve past Clatsop Spit shoals to enter the Pacific Ocean through the jetties in a general southwesterly direction. Although this alignment is dictated in part by harbor facilities at Astoria and vicinity, it appears possible that a change in alignment so as to direct the major river flow due west from Harrington Point past Point Ellice and thence to the entrance might be a factor which could beneficially influence dredging requirements of the entrance channel and could also provide for greater ease in navigation. Such a modification in alignment would, of course, require the maintenance of a channel connecting the realigned main navigation channel with the Astoria Harbor area and adjacent facilities.

Preliminary Analysis of the Problem

As preface to further consideration of phenomena occurring at the entrance to Columbia River, review of the aforementioned difficulties to navigation points to the conclusion that there is a definite problem at this estuary entrance worthy of detailed study in the interest of orderly and timely planning. In such problems, in which complexities are apparently compounded by unknown interrelationships of vaguely defined forces, preliminary analysis, based upon available knowledge and data, frequently serves to lend sound direction to the course of further study.

Past condition surveys represent the source of much of the available information concerning conditions at the mouth of the Columbia River. Examination of these surveys reveals that greater than project depths have prevailed since the beginning of improvement throughout a natural channel, with length ranging from 4 to 7 miles, lying just inside the bar between Clatsop Spit and Sand Island. Attainment of project depths across the bar in 1917 provided a direct connection between this deep channel and the sea, a connection which prevailed until the formation of the inner bar between the jetties in 1952. Concurrently, however, the enlargement of Clatsop Spit pushed this channel against the North Jetty and made it necessary to dredge the face of the spit in order to maintain authorized project dimensions. As indicated previously, this dredging, confined to the Clatsop Spit area, totaled nearly 15,000,000 cubic yards during the period 1939-55. To obtain the increase in project depth to 48 feet, dredging since the spring of 1956 has been necessary on both the outer and inner bars.

Comparisons of computed quantities of material remaining within the authorized channel, shown in Table 1, with reported bin measurements of quantities dredged indicate that recent dredging operations have sometimes been accompanied by a natural scouring action which has greatly assisted the work of the dredges, and at other times by a heavy shoaling within the channel against which the dredges have made little or no headway. Although there appears to be no consistent relationship of scour or shoaling to river discharge, there does appear to be a tendency for the entrance to scour during a rising river condition and to shoal on a falling or almost stationary condition. During the 150-day period, between October 6, 1956 and March 4, 1957, when the river flow was quite uniform and averaged about 140,000 cfs, the entrance

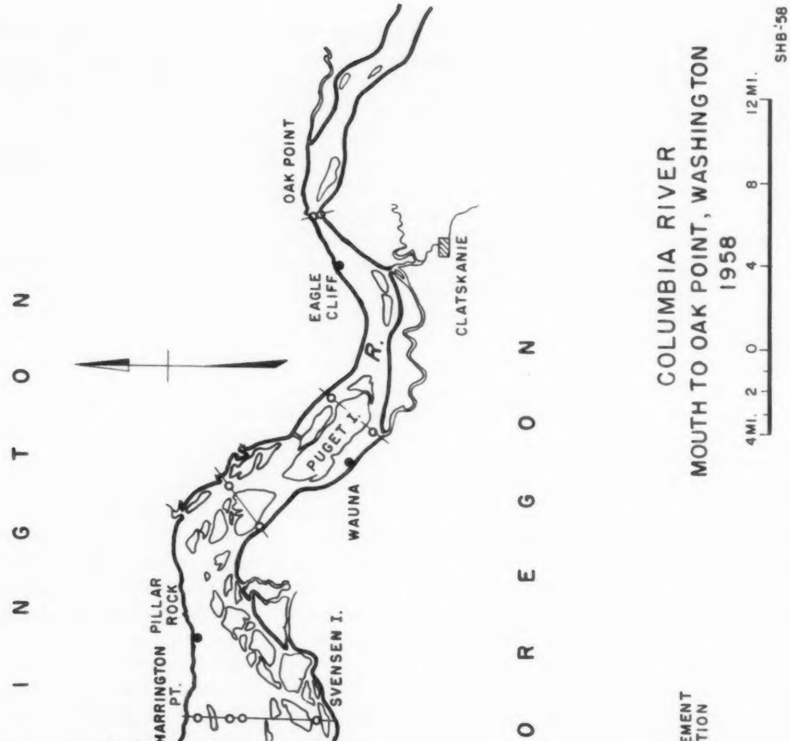
WASHINGTON
OREGON
PACIFIC OCEAN
COLUMBIA R.
CAPE DISAPPOINTMENT
ILWACO
SAND I.
CLATSOP SPIT
TANSY PT.
PT. ELLICE
ASTORIA
TONGUE PT.
HARRINGTON PT.
PILLAR ROCK
SVENSEN I.
WAUNA
PUGET I.
EAGLE CLIFF
CLATSKANIE
OAK POINT
LEGEND
• TIDE GAGE
○ CURRENT MEASUREMENT AND SALINITY STATION
COLUMBIA RIVER
MOUTH TO OAK POINT, WASHINGTON
1958
4 MI. 2 0 4 8 12 MI.
SHB-'58

FIGURE 5

channel shoaled approximately 8,380,000 cubic yards as shown on Table 1. This was partially compensated during the next 90-day period to June 3, 1957, when natural scour amounted to nearly 3,300,000 cubic yards. This shoaling trend was also evident during the winter of 1957-58 when the amount of material in the authorized channel was increased by a total of 5,850,000 cubic yards in the 154-day period between September 18, 1957 and February 19, 1958.

The major portion of the materials dredged at the entrance during 1956 and 1957 has been dumped by the hopper dredges in an area lying about one mile south of the extreme end of the South Jetty. Condition surveys reveal that, although some 14 million cubic yards of material have been disposed in that location since the spring of 1956, very little of this material has remained in that area.

Another phenomenon noted, particularly by condition surveys of April 1957, and later in November of that year, was a formation of waves of material, ranging from 3 to 10 feet in height, along the bottom of the entrance channel on both the inner and outer bars. The degree of wave formation within the entrance channel area appeared more pronounced than in any of the adjacent estuary bottom areas shown by these condition surveys.

As pointed out above, sieve analyses of bin samples of dredged materials indicate that these materials have the general fineness characteristic of sand, although on one occasion material coming within the classification of clay was encountered at the 48-foot depth level. Previous analyses of the composition of these sands, revealing the presence of andesite, granite, and rhyolite, led to an early assumption that their source lies in the upper reaches of the Columbia River. Although these upper reaches may have been the source of a portion of the entrance sands during a previous geological period, the degree of shoaling occurring during the winters of 1956-57 and 1957-58 noted above, when the upland discharge was relatively low, clearly indicates that the upper Columbia River reaches do not now constitute the immediate source or locality from which material is now moved into the channel during, say, a single year. A possible source of these materials may be in the littoral drift offshore along the coast; the evidence provided by the massive build-up of materials behind the North Jetty indicates that these littoral movements, predominately in a southward direction, may be of some significance particularly when influenced by wave action. In addition to littoral drift, it is also possible that offshore bottom areas may be a source of the shoaling materials. Another possibility, although seemingly less plausible, is that these materials may have been deposited in the entrance area by wind action. Complete engineering consideration of the problem should, of course, definitely ascertain the immediate source of the shoaling materials.

In considering the possible forces which may be responsible for the movement of materials within an estuary area, available knowledge limits these forces to river currents, density currents, littoral drift, wind, and waves, or any possible combinations of these forces. River currents have long been recognized as the primary agent responsible for movement of shoaling materials in nontidal streams and in the nontidal portions of tidal streams. However, the action of river currents is known to be modified considerably within an estuary by density currents created by differences in specific gravity of the fresh upland discharge and the salt water of the sea.

A paper by H. B. Simmons[9] points out that engineers have been keenly aware in recent years that the amount of fresh water discharged into an

estuary and the degree to which it mixes with the salt water of the sea are major factors establishing the hydraulic and shoaling regimens of estuaries. With the source of fresh water at the upstream end of an estuary and the source of heavier salt water at the seaward end, each type of water tends to assume, as the result of density differences, a rough wedge shape with the base of the wedge at the source. The sea water entering an estuary does so in this general wedge form, the entering tip of which moves along the bottom. The interface or line of demarcation between the salt and fresh water may vary from well defined to almost obscure depending on the degree of mixing of the salt and fresh waters. The ratio of mean upland discharge over a mean tidal cycle interval to the mean tidal prism, as measured by the volume of water flowing into an estuary from the sea during an average flood-tide period, may be used to roughly define three broad categories of mixing: highly stratified, when the ratio is 1.0 or greater; partly mixed, when the ratio is in the order of 0.25; and well mixed, when the ratio is appreciably less than 0.1.

Distribution of current velocities in a highly stratified estuary, such as the Southwest Pass of the Mississippi River, shows that upstream from the tip of the salt-water wedge the direction of flow throughout the entire depth is toward the sea at all times with a vertical velocity distribution similar to that of an upland stream. Farther downstream the flow in the fresh-water strata is still toward the sea, but the direction of flow in the underlying salt water is upstream at all times. The cause of the continuous upstream flow within the salt-water wedge is constant erosion of salt water from the interface by the outflowing fresh water. The salt water thus eroded must be replaced to maintain stability and, accordingly, a continuous upstream current is generated beneath the interface. In this type of estuary, rapid shoaling usually occurs in the region of the salt-water intrusion tip. The heavier particles of material, which are rolled or pushed along the river bed by the outgoing fresh water, come to rest as soon as the tip of intrusion is reached. The lighter particles, transported largely in suspension in the fresh water, gradually fall through the interface along the length of intrusion and are transported upstream by salt water currents to the vicinity of the tip. Flocculation of dissolved materials in the fresh water may also occur along the interface and form part of the shoaling mass.

Available records indicate that upland discharge into the Columbia River estuary has ranged from an extreme low of 60,000 cfs to a maximum of about 1,300,000 cfs. The ratios of these discharges to measurements of average flood-tide flows shown in Table 2 indicate that the mixing of fresh and salt water within this estuary may assume characteristics of the highly-stratified estuary during the spring freshets and of the partly-mixed estuary during normal and low-flow periods. Evidence that there exists pronounced salinity differences between the top and bottom layers of water of the estuary is available from observations made by Oregon State College and Oregon Fish Commission, shown in Tables 3 and 4, on July 25 and 26, 1955, at which time the upland discharge was about 360,000 cfs. This finding was confirmed by observations of the U. S. Hydrograph Office made in 1954 and 1958 as noted above. Earlier observations in 1936 indicated that the upstream penetration of the salt-water wedge may range from 11 to 15 miles.

As the result of a recent preliminary reanalysis of the findings of the Columbia River current survey of 1932, Simmons, in a later paper(10) stated that the effects of density are of major importance with respect to shoaling at

the Columbia River entrance. While recognizing that more detailed studies later may show that certain of the resulting indications are not completely valid, this preliminary reanalysis, made for three different upland discharges, revealed that at a point 5 miles above the outer ends of the jetties, the flow at 90 per cent depth was predominantly upstream during an upland discharge of 127,000 cfs and essentially in balance (50 per cent upstream and 50 per cent downstream) during upland discharges of 320,000 and 584,000 cfs. Consideration of only those current velocities in excess of 0.6 feet per second, competent for movement of sand with median grain size of 0.4 to 0.5 mm (bin sample analyses noted above indicate median grain sizes of 0.25 mm on the inner bar and 0.20 mm on the outer bar) resulted in a marked increase in the upstream predominance of effective flow at 90 per cent depth for an upland discharge of 127,000 cfs, a change from a balanced flow condition to one of upstream predominance for a discharge of 320,000 cfs, and no change from a balanced flow condition for a discharge of 584,000 cfs. Consideration of only those current velocities in excess of 1.5 feet per second, at and above which velocity the rate of sand movement should be appreciable, resulted in a further increase in flood predominance at 90 per cent depth for upland discharges of 127,000 and 320,000 cfs, and a change from a balanced flow condition to one of definite upstream predominance for a discharge of 584,000 cfs. The above available evidence and reanalysis of the 1932 current survey, together with information not susceptible to complete check at this time, point to great probability that density currents, created by salinity differences, constitute one of the major, if not the most dominant, forces controlling the shoaling regimen of the Columbia River entrance.

Except for its possible effect in creating the large build-up of materials behind the North Jetty, very little information is available concerning the nature or extent of offshore littoral drift movements at the Columbia entrance. Considering the great mass of materials now trapped and consolidated behind that structure, it may reasonably be assumed that an appreciable amount of material is moved by littoral forces across the entrance area. It may also be reasonable to expect that a portion of this material may be carried through the entrance by flood tides and waves and deposited so as not to be removed by ebb tides. Clatsop Spit and Sand Island might have had their origins in such action and study to determine the mineral and chemical composition and particle-shape characteristics, as well as mechanical analyses, should be made of the materials forming these geographic features to ascertain whether or not they are formed of littoral materials.

Although the existing project provides for the construction of spur Jetty "B" to reduce maintenance costs for dredging and to provide protection against damage to the shorelines and main jetties from scour which may be caused by a change in currents resulting from the deeper channel, preliminary analysis of the shoaling problem raises doubt concerning the effectiveness of that structure, particularly when viewed in the light of the probable forces controlling the shoaling regimen of the entrance area. Considering the spur jetty structure as a contraction work whose greatest effectiveness would occur during the high-flow periods, it is doubted that Jetty "B" would be instrumental in reducing, or affecting to any material degree, heavy shoaling such as occurred during the relatively low-flow periods in the winters of 1956-57 and 1957-58. These factors, as well as the cost of the structure, currently estimated at $5,944,000, and the possible economies which might be effected by alteration of its design in the light of a detailed appraisal of the

forces to be encountered, point to the advisability of a more considered approach to solution of the problem.

Preliminary analysis indicates that such an approach should include a comprehensive study of all phenomena susceptible of contributing to shoaling within the Columbia River entrance area, of which the phenomena of density currents and littoral drift appear to be the primary and controlling agents. Such a study should be directed toward answering the following questions:

a. What and where are the immediate sources of shoaling materials?
b. What are the effects of density currents and littoral drift on shoaling?
c. If density currents and littoral drift are primary agents for moving materials from the sea into the estuary, what counteracting forces create the balance of conditions in the entrance area?

Plan of Action

In the light of experience gained in the study and improvement of tidal entrances elsewhere in the United States, the problem at the Columbia River entrance was presented to the Committee on Tidal Hydraulics of the Corps of Engineers early in 1956 and as the result of that presentation the Committee recommended that a program of prototype measurements be undertaken as soon as possible with the view to investigating the problem in a hydraulic model at an early date.

Available data which might be useful in a model study were examined, and based on this examination, the Director, Waterways Experiment Station, Vicksburg, Mississippi in March 1957, recommended a program of hydraulic and salinity measurements required for the design and verification of a hydraulic model. This program consists of three cycles of measurements to obtain data for conditions of low, normal, and high river discharge, each cycle to include observations of current direction and velocity for a continuous full tidal period of about 25 hours at 19 stations located on six ranges as shown in Fig. 5. At each station measurements will be made on 30-minute intervals at the surface and bottom and at the intervening quarter points of depth. Simultaneous observations will be made initially at one station on each range for a continuous period of about 25 hours to establish the relationship between ranges followed by simultaneous observations at all stations on each range until all observations have been obtained. Salinities will be observed at hourly intervals concurrently with the observations of current velocities and directions. If practicable, an attempt will be made to obtain a few measurements of current velocities, current directions, and salinities on a range between the jetties. Considering the equipment which may be available for this purpose, it is anticipated that a complete measurement cycle can be completed in six days.

It is planned to undertake the above program of prototype measurements as soon as necessary instrumentation, operating personnel and equipment, and funds can be made available. It is recognized that, even with an expedited program of prototype measurements followed by the immediate design, construction, and verification of a hydraulic model, it may be a number of years before the forces that control the regimen of the Columbia River entrance may be accurately defined and suitable corrective measures designed and constructed. In the meantime, it is anticipated that sufficient maintenance funds

will be made available to provide, by dredging, existing project depths over usable widths throughout the entrance channel.

CONCLUSION

An engineering approach to solution of the problem at the Columbia River entrance should include a qualitative definition of forces controlling the regimen of the estuary area, such as may be obtained by means of a prototype measurement program, together with a quantitative analysis of the magnitudes and effects of these forces as may be provided by means of a hydraulic model study. Such steps in the past have led to the solution of complex tidal problems in the Delaware River, Charleston Harbor, Savannah Harbor, and other improved river estuaries by revealing the structural measures necessary to make effective use of the forces controlling the particular estuarine regimens involved and similar steps should be no less instrumental in pointing to a solution of the Columbia River entrance problem. Even though a model investigation might conclusively show that solution of this problem lies not in additional structural measures but in regular channel dredging, this investigation would be of inestimable value in that it would definitely eliminate further structural measures from future consideration. In the light of such a finding, substantial savings could be effected by limiting future expenditures to maintenance dredging operations only.

ACKNOWLEDGMENT

Although solution of the problem at the Columbia River entrance has not yet been obtained, the services of a great many engineers over the past century have contributed toward achievement of that goal. This paper is presented in full recognition of their efforts.

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AIR MODEL STUDIES OF HYDRAULIC DOWNPULL ON LARGE GATES[a]

W. P. Simmons, Jr. (AM)[1]

ABSTRACT

Air model studies were used to determine the hydraulic downpull forces on a large fixed-wheel gate and on a slide gate. The applicability of air tests for this work is discussed, and the method of applying the data is detailed. Data are presented that will be helpful in evaluating downpull forces on other similar gate structures.

INTRODUCTION

The use of air models for studying hydraulic problems has repeatedly proved to be a reliable and accurate expedient. The advantages of air tests in place of hydraulic tests have been thoroughly discussed by Hunter Rouse(1) and others, and many examples of the use of these air models are given by J. W. Ball and D. W. Appel.(2) Another unusual use of air models is presented in this paper, wherein studies of hydraulic downpull forces on a large fixed-wheel gate and on a high-pressure slide gate are discussed.

The downpull forces being considered here are those forces produced when flow occurs beneath the gate leaf, thereby reducing the pressures on the bottom of the leaf relative to the pressures on the top. This pressure difference, acting on the cross-sectional area of the leaf, produces a downpull force which must be considered in addition to frictional forces and the leaf weight in designing the stem and hoist. The method used for obtaining the downpull was to determine the unit pressures acting on the top and bottom surfaces of the gate leaves, and applying these pressures to appropriate areas to compute the downpull forces. No measurements of stem loads were made.

Note: Discussion open until June 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1903 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. HY 1, January, 1959.

a. Presented at the June 1958 ASCE Convention in Portland, Oregon.

1. Hydr. Research Engr., Hydr. Structures and Equipment Section, Hydr. Lab., Bureau of Reclamation, Denver, Colo.

When the downpull studies were first considered, it was taken for granted that model studies would be made with water. But it was later recognized that air model studies would be equally satisfactory and that they would be less expensive and time consuming. The method of testing with air would be identical to that for water because the air velocities would be maintained below about 250 feet per second, and incompressible flow equations could be used.(2) The degree of error would not exceed about 5 percent at a velocity of 250 fps and would decrease rapidly with decreases in velocity. The velocities actually used were 64 fps or less, and the error was about 1 percent. Wetting would not be a problem in an air model and, therefore, untreated wood and metal surfaces could be used. The construction could be relatively lightweight because the density of air is low and the weight and pressure forces are small. The supply and exhaust systems posed no problems because the atmosphere served as both reservoir and receiver.

The argument in favor of hydraulic models was that by their use free water surfaces could be obtained. With air models discharging into the atmosphere no free surfaces are possible. With either of the two gates under consideration free surfaces will exist during free discharge operation.

This limitation was not as great a handicap for the air models as was first thought. In the case of the fixed-wheel gate, the maximum downpull occurs when there is no free surface and where back pressure or submerged conditions exist. Thus, only tests with submerged flow were required. These are accurately reproduced in an air model.

In the case of the slide gate, the downpull forces were desired at both free discharge and submerged conditions. After study of the leaf shape, it was found that the pressure acting on the leaf could be analyzed for either free discharge or submerged conditions by proper treatment of the data obtained from an air model. The treatment is discussed in detail later in this paper.

Gate Structures

The first gate studied was a fixed-wheel gate designed for the power conduit at Glendo Dam, Wyoming (Figure 1). This gate has a leaf 3.3 feet thick, 16.5 feet wide, and 21.0 feet high. It is located near the entrance of the 21-foot-diameter outlet and power tunnel and normally remains fully open. When a regular closure is required, it is made with no flow taking place and with the pressures upstream and downstream from the leaf balanced. Emergency closures with unbalanced pressures may also be made. Sustained operation with unbalanced pressures will occur during the tunnel filling period when the gate will be used as the flow regulator at openings of 3 to 6 inches.

The control point on this fixed-wheel gate is at the upstream bottom edge of the leaf. When the gate is controlling the flow, the pressures on the leaf bottom will be less than the pressures on the leaf top and less than in the tunnel just downstream. This occurs because as the flow passes beneath the leaf, the flow velocity— and hence the kinetic energy—becomes high. Simultaneously, and because the total energy in the water remains essentially constant, the piezometric pressures become low.

The pressures on the leaf bottom were of greatest interest during the tunnel filling period when the gate openings were very small. At these small openings, the ratio of opening height to gate thickness was small, so that in effect, the opening under the leaf approximated a "short tube," or more

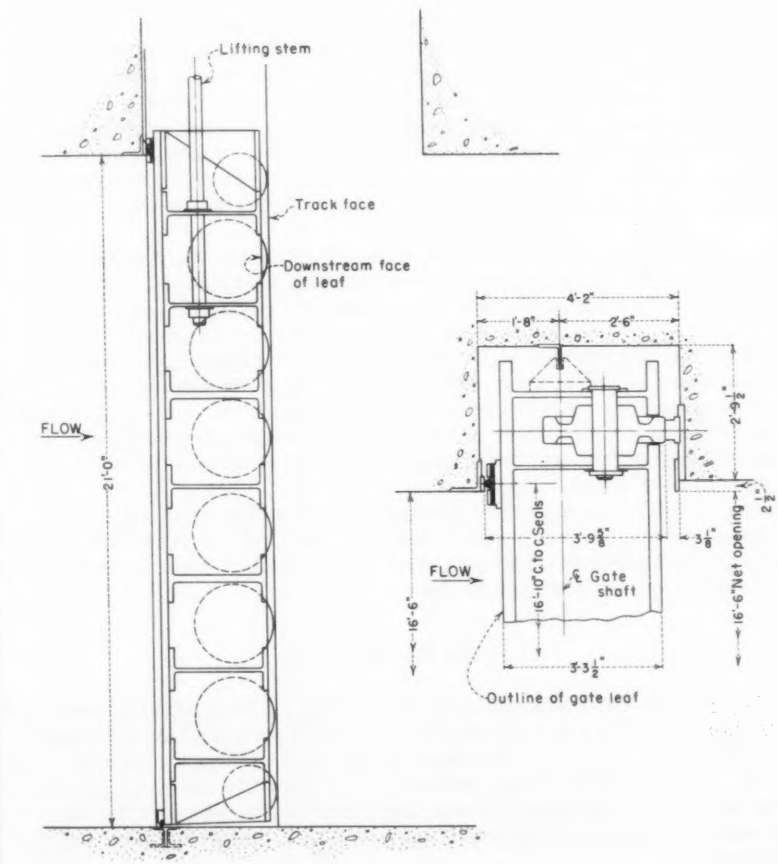


FIGURE 1 - 16.5' x 21' FIXED-WHEEL GATE
GLENDO DAM, WYOMING

specifically, a "short slot." As the flow passed beneath the leaf, it experienced it's minimum cross section at the vena contracta, and then tended to expand and occupy most of the section between the conduit floor and the web of the bottom beam at the downstream face of the leaf. This action could prevent aeration between the leaf and the jet during free discharge operation, and would restrict the relief afforded by circulation of water during submerged operation. Thus, lower pressures than usual would occur on the entire leaf bottom at very small gate openings, and the downpull forces would be extremely large. When the gate opening increases, the jet contraction and subsequent expansion changes and more space is left between the jet and the downstream web. The fluid is readily able to move upstream through the space to relieve the low pressures on the gate bottom. When the gate reaches the full open position, the pressures beneath the leaf are about the same as those in the tunnel and on top of the leaf. At this point, the downpull is quite small.

The second gate studied was the slide gate design now being extensively used by the Bureau of Reclamation. This gate is a rectangular, downstream seal, high capacity slide gate designed for use as a regulator under high heads (Figure 2). The principal features of the gate are the small slots, the outwardly offset downstream slot corners followed by gradually converging side walls, and a leaf with a flat upstream face and a 45^{o} sloping bottom. This gate was developed to meet the requirements of the outlet works at Palisades Dam where large flows of water at heads up to 240 feet are controlled. The gate has since been included in the designs of several outlet structures with heads as high as 373 feet.

The magnitude of the hydraulic downpull forces on the fixed-wheel gate and on the slide gate were desired so that adequate, but not excessive, hoists and handling equipment could be provided. Model studies were made on the gates to evaluate these downpull forces.

The Models

Air was used as the flowing fluid in the models. A centrifugal blower drew air from the atmosphere and forced it through a 10-inch pipeline 6.4 diameters long, and then through the test section and back to the atmosphere (Figure 3). The rate of flow was measured by a flat plate orifice at the entrance of the 12-inch-diameter blower inlet line. All of the parts for the models were of simple and inexpensive construction, and were fabricated in a short time.

The test section for the fixed-wheel gate consisted of a plywood conduit 16.67 inches high (Figure 4). The conduit was 8.25 inches wide upstream from the gate leaf, and 8.50 inches wide downstream from the leaf. The 2.82-inch-thick leaf was made of 12-gage sheet metal and contained 10 piezometers on the bottom along or near the conduit center line. Piezometers in the tunnel roof upstream and downstream from the leaf were used to obtain the pressure drop across the gate. The slots were simplified and represented the width but not the depth of the Glendo slots. The position of the gate leaf could be observed through transparent plastic windows that formed the outer slot walls.

The test section for the slide gate was 16.67 inches high and 8.25 inches wide (Figure 5). The 2.48-inch-thick leaf was made of heavy gage sheet metal

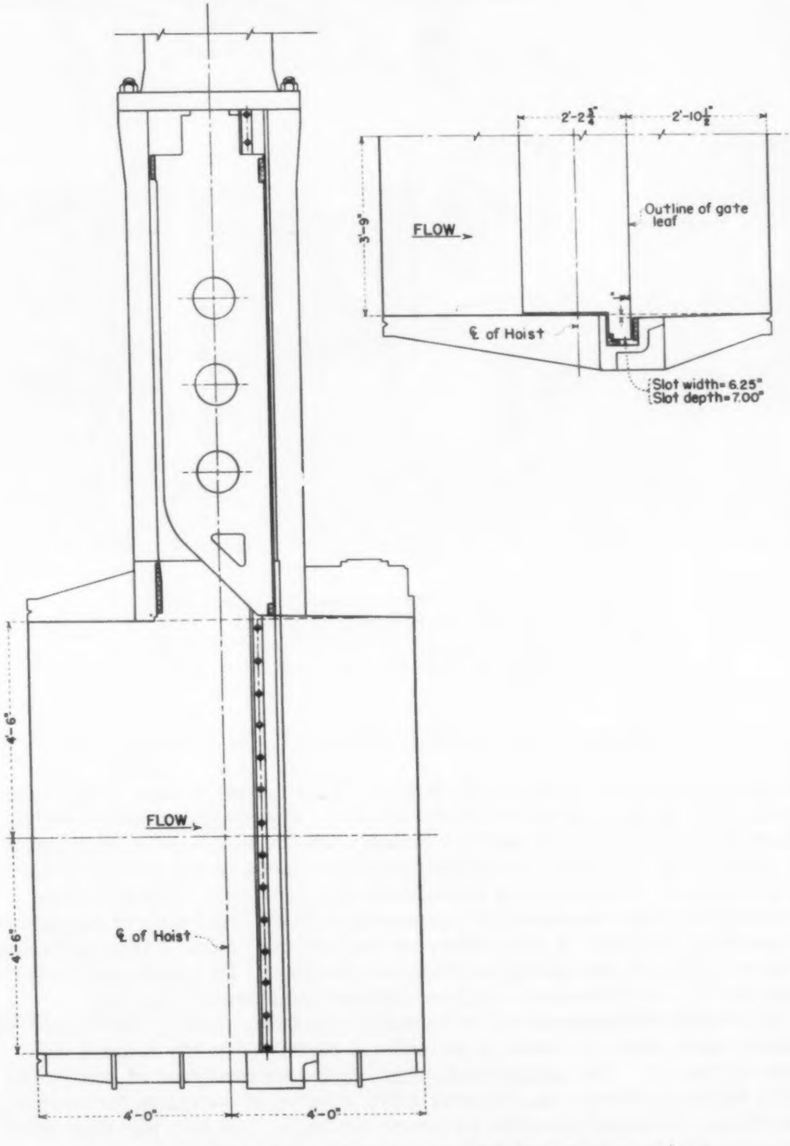
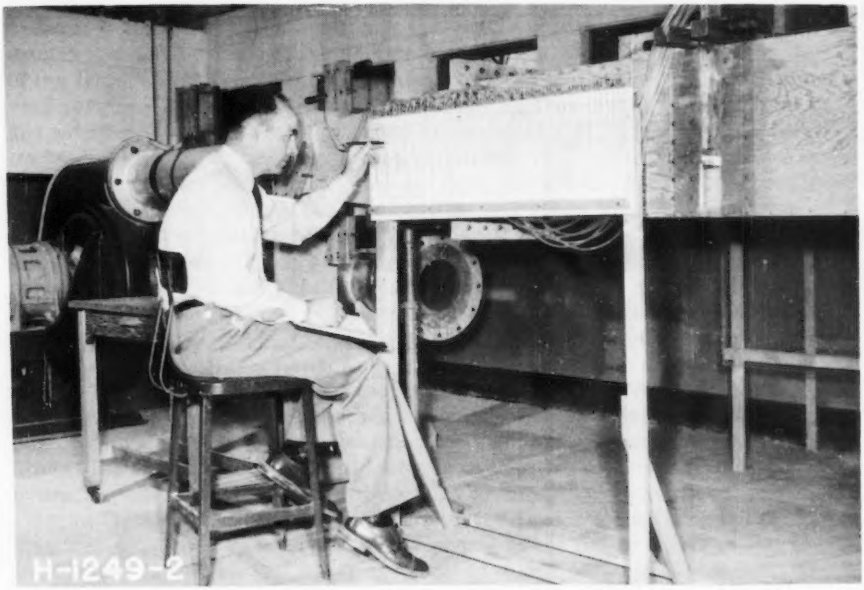


FIGURE 2 - 7.5' x 9.0' HIGH HEAD SLIDE GATES
PALISADES DAM, IDAHO



Air was drawn from the atmosphere into inlet orifice at center by centrifugal blower at left. Flow passed through a 10-inch-diameter pipe section, then the test section, and back into the atmosphere.

FIGURE 3 - AIR MODEL AND TEST FACILITIES

and was supported in slots 0.64 inch wide. Three rows of eight piezometers were placed on the leaf bottom so the pressure distribution could be determined at distances of 0.18 and 1.10 inches from the sides of the leaf, and on the center line. An additional piezometer was placed on the bottom of one of the projections of the leaf that extends into the gate slots. The upstream pressure head was measured by a piezometer in the right wall 19 inches upstream from the leaf. A piezometer on the roof center line 1 inch upstream from the leaf gave the approximate bonnet pressure. No bonnet was included in the model, and there were no piezometers downstream from leaf.

All pressure measurements were made with water-filled U-tubes, and the readings were made in tenths of an inch and estimated to the nearest hundredth of an inch. The test procedure for each gate consisted of setting the leaf to the desired position, allowing a few minutes of operation for conditions to stabilize, and then taking the pressure readings. The leaf was then set to the next desired position and the procedure was repeated.

Investigations

The downpull forces on the gate leaves were determined by measuring the pressures acting on the top and bottom surfaces of the leaves, and not by

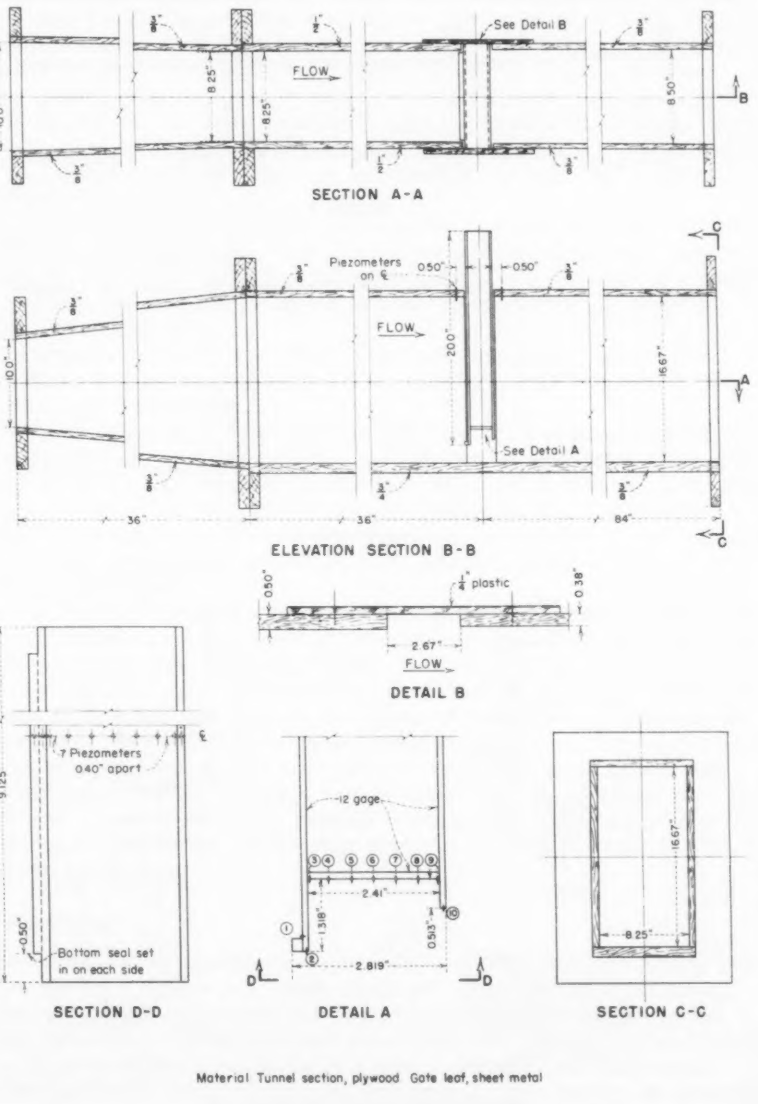
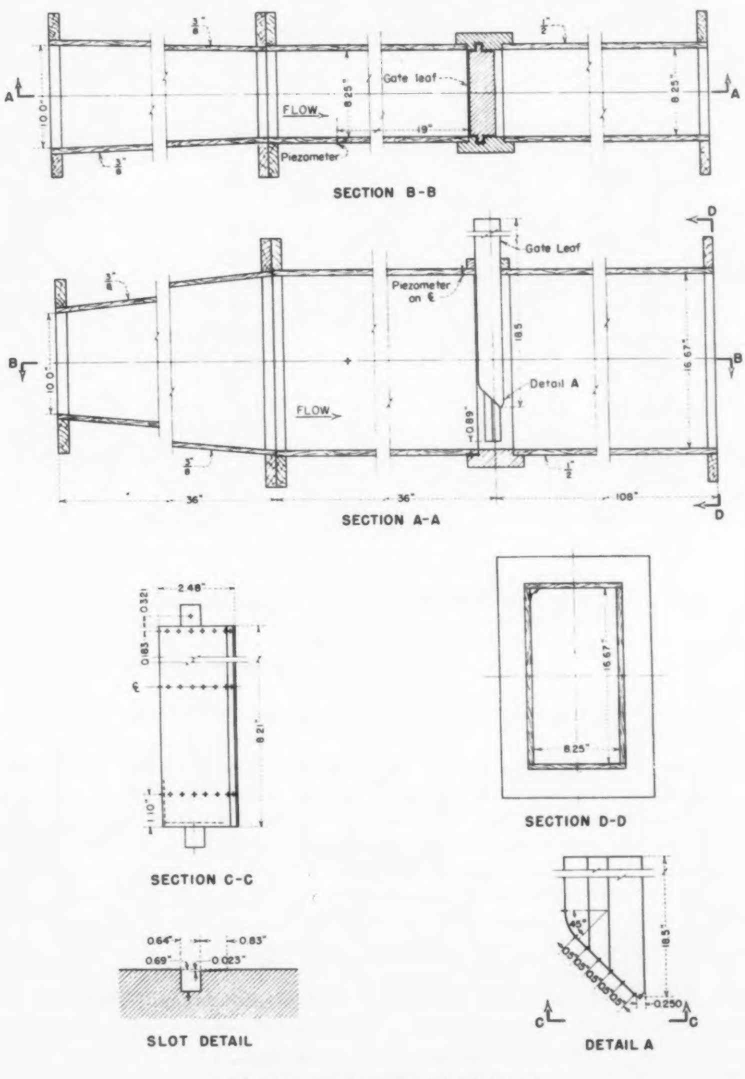


FIGURE 4 - FIXED-WHEEL GATE TEST SECTION

AIR MODEL



Material Tunnel section, plywood. Gate leaf, sheet metal

FIGURE 5 - SLIDE GATE TEST SECTION

AIR MODEL

measurement of stem loads by strain gages, spring scales, weights, or other devices. Previous experience at the Bureau of Reclamation has shown that the forces determined by pressure distribution may be more accurate than forces determined by stem load measurements because of the effects of frictional factors, vibration, and leaf movement. Proper clearances between parts are of particular importance, and it is often difficult to predict what these clearances will be, or to produce them in the model. When using the pressure-area method, care must be taken to insure getting pressure readings in all regions which can effect the downpull on the leaf, and to apply these pressures properly.

In the case of the Glendo fixed-wheel gate, the only condition where downpull will be an important factor is during submerged operation when gate openings are small relative to the leaf thickness. In this case the only pressure relief obtained beneath the leaf is that due to circulation of water between the leaf and the jet.

The pressures on top of the leaf can vary from essentially atmospheric when the backwater is insufficient to fill the tunnel, to high values when the downstream tunnel is flowing full.

When no backwater is present and the gate opening is appreciable, the jet will pass beneath the leaf and clear the downstream web enough to allow free aeration. Thus, the pressures under the leaf will be about atmospheric. The pressures above the leaf will also be about atmospheric because the gate seals are on the upstream face and the bottom surface is open to the tunnel pressure at the downstream face (Figure 1). There is little or no downpull under these conditions.

In the slide gate, downpull will be an important factor through a wide range of openings for both free discharge and submerged operation. The maximum downpull can occur with submerged operation because subatmospheric pressures can exist on the flat-bottomed portion of the leaf during this operation. In contrast to this, the lowest pressures on the flat-bottomed portion of the leaf during free discharge operation are about atmospheric. The bonnet pressures will be high at small gate openings for either free discharge or submerged operation because this type of gate seals on the downstream face of the leaf, and is open to the tunnel pressures at the upstream face (Figure 2). The high bonnet pressures are the main factor in the high downpull forces produced on these gates.

Fixed-Wheel Gate

In the simplified air model equivalent values of head differential that will occur across the full-size gate could not be set. But, as in a water model, the pressures on and across the leaf varied in accordance with the model heads and velocities. Thus, prototype pressures were obtained by multiplying model pressures by the ratio of the computed prototype head differential across the gate to the measured model differential. To compute the prototype head drop, it was assumed that the 21-foot-diameter tunnel was ruptured at the power bifurcation, and that the reservoir was at the maximum water surface elevation of 4669.0, producing a head of 178 feet. The head loss through the intake structure and tunnel with the gate 100 percent open limited the maximum flow to 21,500 cfs. As the gate closes to reduce the flow, the tunnel losses become smaller and the head drop across the gate becomes larger. At very small openings nearly the full shut-off head of 178 feet is acting on

the gate. The relation of prototype head drop to gate opening is shown in Figure 6A.

The model was tested at a number of gate openings, and the pressure drop across the gate and the pressures on the bottom of the leaf were measured at each of the openings. These measured pressures are shown in Table 1. It is interesting to note that the pressure just above the lower seal (Piezometer No. 1) is almost the same as the pressure acting on the roof at the leaf, and hence on the upper seal. The upper seal has the same plan area as the lower one, and therefore the upward force on the upper seal is the same as the downward force on the lower seal. These forces balance one another; thus Piezometer No. 1 is not used in the downpull calculation. From these data, the prototype downpull forces were calculated as follows, using as an example a 3-inch prototype gate opening.

Model pressures in inches of water were:

Upstream head	Downstream head	Seal	Gate web (average)	D. S. lip
+8.07	-0.09	-1.25	-0.51	-0.52

Prototype head drop (from Figure 6A) = 178 feet

$$\text{Factor for conversion} = \frac{178}{8.07 + 0.09} = 21.80$$

Downstream head = -(0.09 (21.80) = -1.96 = head on top of leaf

Under the seal = -(1.25) (21.80) = -27.21

Under the web = -(0.51) (21.80) = -11.12

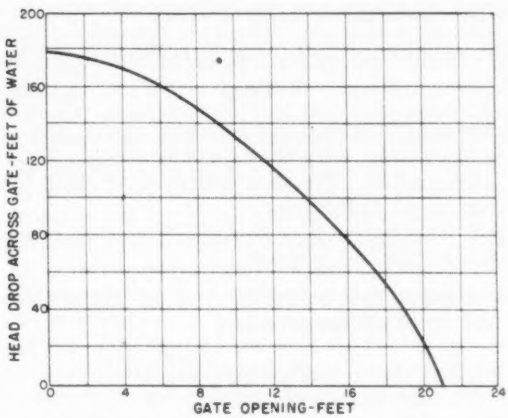
Under the DS lip = -(0.52) (21.80) = -11.34

If it is assumed that the pressure distribution at any section along the leaf is the same as the measured distribution at the center line section, the downpull may be found by multiplying the pressure difference between the gate top and bottom by the specific weight of water and by the area of the gate section (Figure 6B) on which the pressure difference acts.

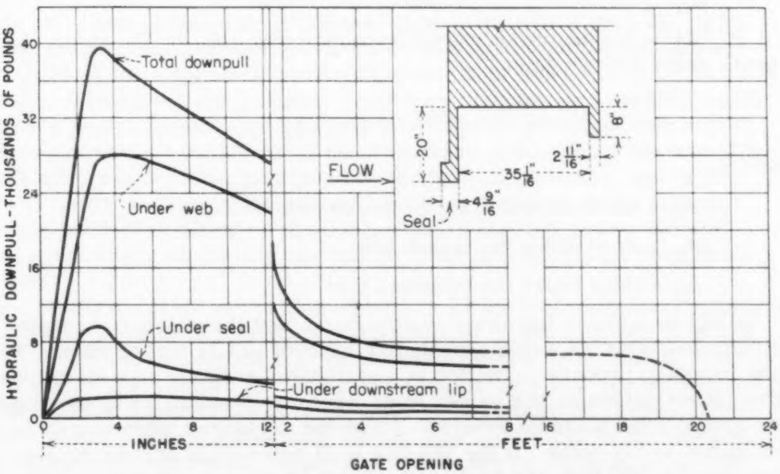
Table 1

MODEL PRESSURES (IN INCHES OF WATER) ON GLENDO FIXED-WHEEL GATE

	Piezometers											
Gate opening	1	2	3	4	5	6	7	8	9	10	U.S.	D.S.
0'-1.5"	8.08	-0.72	-0.27	-0.26	-0.28	-0.27	-0.27	-0.26	-0.26	-0.26	8.06	-0.04
0'-3"	8.06	-1.25	-0.50	-0.51	-0.53	-0.52	-0.52	-0.52	-0.49	-0.52	8.07	-0.09
0'-4.5"	8.46	-0.97	-0.51	-0.48	-0.47	-0.49	-0.54	-0.50	-0.50	-0.51	8.53	-0.07
0'-6"	8.38	-0.81	-0.53	-0.52	-0.56	-0.56	-0.58	-0.53	-0.50	-0.54	8.58	-0.10
0'-9"	7.57	-0.55	-0.47	-0.46	-0.49	-0.48	-0.49	-0.48	-0.45	-0.49	7.74	-0.11
1'-0"	7.14	-0.40	-0.34	-0.34	-0.35	-0.35	-0.37	-0.36	-0.33	-0.41	7.28	-0.03
1'-6"	8.90	-0.36	-0.34	-0.35	-0.37	-0.37	-0.35	-0.34	-0.34	-0.35	9.20	-0.05
2'-0"	8.97	-0.05	0.07	0.06	0.02	0.04	0.05	0.08	0.11	-0.01	9.36	0.15
3'-0"	8.65	-0.66	-0.55	-0.56	-0.60	-0.55	-0.55	-0.50	-0.47	-0.69	9.13	-0.49
5'-0"	6.95	-1.92	-0.79	-1.80	-1.87	-1.85	-1.86	-1.78	-1.72	-1.86	7.61	-1.79
6'-0"		-2.01		-1.94		-2.00		-1.97		-1.99	6.62	-1.86
10'-0"	5.56	2.19	2.28	2.21	2.23	2.24	2.25	2.25	2.25	2.25	5.60	2.33
15'-8"	1.75	1.66	1.63	1.67	1.64	1.64	1.64	1.67	1.65	1.62	1.75	1.63



A. HEAD DROP ACROSS GATE AS GATE IS CLOSED FOLLOWING A PENSTOCK BREAK 1865 FEET DOWNSTREAM. RESERVOIR ELEVATION 4669.0



B. DOWNPULL WHEN GATE CONTROLS FLOW AT RESERVOIR ELEVATION 4669.0

FIGURE 6 - HEAD DROP ACROSS, AND DOWNPULL FORCES ON GLENDO 16.5' x 21.0' FIXED-WHEEL GATE

DATA FROM AIR MODEL

Seal	= (27.21 - 1.96) (62.4) (16.5) (0.3802)	=	9,900 pounds
Web	= (11.12 - 1.96) (62.4) (16.5) (2.9282)	=	27,610
DS lip	= (11.34 - 1.96) (62.4) (16.5) (0.2240)	=	2,160
	Total		39,670 pounds

A plot was made of the computed prototype downpull forces acting on the seal, under the lower web, and under the downstream lip for a wide range of gate openings (Figure 6B). The total downpull, which is the sum of the individual downpulls, is also shown.

This downpull information may be applied to other geometrically similar installations by the following method:

A. Find the downpull that would occur at the desired gate openings with the head drop for the proposed installation:

$$DP_2 = DP_1 \times \frac{H_n}{H_g}$$

where

DP_1 = Downpull, Figure 6B

DP_2 = Downpull with H_n

H_g = Head drop across Glendo gate, Figure 6A

H_n = Head drop for proposed installation

B. Obtain the desired downpull by multiplying DP_2 by the ratios of the areas under the two gates:

$$DP_3 = DP_2 \times \frac{A_n}{A_g}$$

where

DP_3 = downpull under the proposed gate with H_n

A_g = area under the tested gate

A_n = area under the proposed gate

It will be noted in the sample computation that the pressure beneath the bottom seal reached a dangerously low value of -27.21 feet of water. This is low enough to produce cavitation in installations at fairly high elevations. Even at elevations as low as sea level, pressure fluctuations of only moderate proportions would be enough to lower the pressures momentarily to where cavitation would occur. If the 178 foot head differential across the gate were increased to 200 feet, the seal pressure would become -30.60 feet and cavitation could extend over much of the seal. At a 250-foot head differential, cavitation pressures would exist for gate openings from about 2 to about 4 inches. For the conditions where cavitation is general, downpull on the seal is computed by applying cavitation pressures to the area under the seal.

Slide Gate

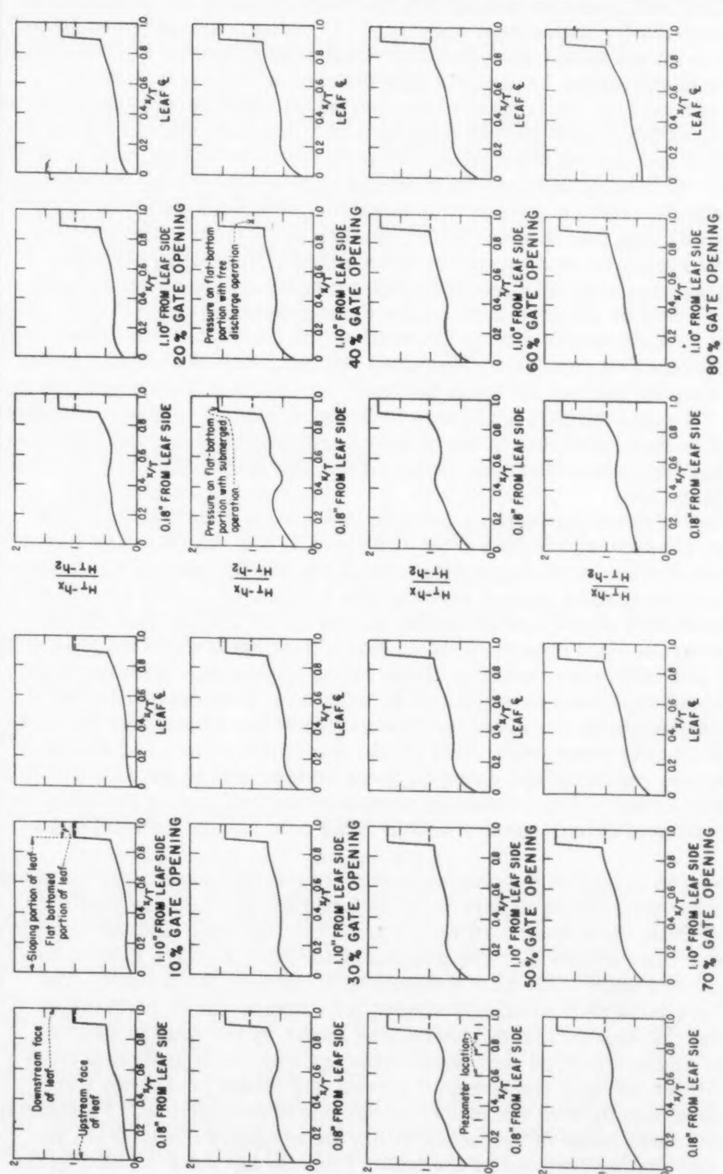
It has been previously noted that a model that uses air as the flowing fluid and that discharges into the atmosphere is operating under submerged

conditions. In the case of the slide gate, the flow patterns along the upstream face of the leaf and along the sloping bottom of the leaf will be the same for free discharge and for submerged operation. Pressures measured on these components in an air model are therefore equally applicable to free discharge and submerged conditions. A distinct flow difference will exist, however, along the horizontal or flat portion of the leaf which lies downstream from the abrupt ending of the slope. During free discharge the flow will spring clear of this surface and the pressure on it, and on any seal extension, will be about atmospheric. During submerged operation an eddy of the flowing fluid will be in contact with the surface and the pressures will be a function of the head, the gate opening, and the effective back pressure on the gate. The lowest pressure that can occur on the surface when water is flowing is the vapor pressure of water, about -30 feet gate. The pressure under a seal extension, if one is used, will be affected in about the same manner.

Practical considerations dictate the width of the flat on the leaf bottom and on any seal extensions required. Therefore the geometry of the gate bottom may not be exactly similar for gates designed for different sized installations and heads. To make the data applicable to these variations in design the data was reduced to that applicable to the sloping portion of the leaf, and that applicable to the flat-bottom portion, including the slot projections and seal extensions (Figure 8A).

The method of determining the slide gate downpull was as follows: The model bonnet pressures, and hence the pressures acting downward on top of the leaf, were divided by the head differentials across the gate. This dimensionless ratio was plotted against percent gate opening in Figure 8B. Similarly, the pressures acting upward on the sloped portion of the leaf bottom were divided by the total heads producing flow. This ratio was also plotted in Figure 8B. The difference between these ratios, at any gate opening, multiplied by the prototype head producing flow, equals the prototype differential head acting downward on the cross-sectional area of the sloped portion of the leaf. This head difference, multiplied by the appropriate area, and by the density of water, results in the downpull force on this part of the leaf. To this force, the forces acting on the flat portion of the leaf and on the slot projections and seal extensions are added. This sum equals the total downpull force.

The pressures acting on the sloping portion of the leaf were investigated in detail. Three rows or piezometers were included on the leaf, one 0.18 inch from the leaf side, the second 1.10 inches from the leaf side, and the third in the leaf center line (Figure 5). The pressures for each row are plotted nondimensionally in Figure 7 for gate openings of 10 through 80 percent. The pressure at any point on a similarly shaped leaf bottom can be determined by multiplying the appropriate dimensionless factor by the head producing flow (H_T-h_2) and subtracting this result from the total head upstream from the gate. For determining the downpull forces, the model pressures were replotted dimensionally with the forces being considered vertical. Planimeter measurements were made of the areas within the pressure distribution envelopes for the sloping portions of the leaf. These areas were divided by the 2.23 inch thickness of the leaf above the sloped bottom to give an average pressure value at each piezometer row. The average pressures obtained for the row 0.18 inch from the leaf side were assumed to act from the leaf sides to stations 0.50 inch from the sides. The pressures for the row 1.10 inches from the side were assumed to act from the 0.50 inch stations to stations 2



H_T = Total head upstream from gate
h_x = Pressure head at leaf piezometers
h_2 = Pressure head downstream from gate

x = Distance from upstream face of leaf
T = Leaf thickness
Model leaf 8.21" wide x 2.48" thick

FIGURE 7-PRESSURES ON BOTTOM OF SLIDE GATE LEAF

DATA FROM AIR MODEL

inches from the sides. The pressures at the center line piezometers were assumed to act over the remaining width of the leaf. The products of these average pressures and appropriate areas were added together for each gate opening. These pressure-area summations were divided by the projected area of the sloping portion of the leaf to give the equivalent single pressure values that can be assumed to act over the sloped area of the leaf at each gate opening. These pressure values, divided by the total head producing flow, establish the curve plotted in Figure 8B. At zero opening, the average pressure acting upward on the plan area was the same as the total head. It decreased to a minimum at 70 percent open and increased again at 80 percent open. No tests were made at openings above 80 percent because the downpull would be small.

Free discharge conditions

In the full-sized slide gates the back of the leaf carries a seal that is in contact at all gate openings with a seal plate in the bonnet. There can be no flow or appreciable leakage between the back of the leaf and the bonnet, and only a little flow down the gate slots. Thus the bonnet pressure is about the same as the stagnation pressure just ahead of the leaf. The model pressure measurements, based on the piezometer in the top of the conduit 1 inch ahead of the leaf showed that without submergence the bonnet pressure was the same as the total head in the conduit at zero gate opening, and that it decreased with respect to the total head as the gate was opened (Figure 8B).

The difference between the bonnet pressure and the average pressure over the sloped bottom was a maximum at an opening of about 45 percent and the difference did not exceed about 53 percent of the total head (Figures 8B and C). The net downpull on the flat bottom, the slot projections, and any seal extensions can be determined from their areas and the difference between the bonnet pressures and atmospheric pressure. The total downpull will be the sum of the individual downpull forces acting on the various portions of the leaf.

A sample calculation of the downpull forces follows:

Assume: Free discharge conditions, gate leaf 6.0 feet wide, 2.0 feet thick, 3-inch flat on bottom, 6- by 6-inch slot projections, no seal extension (Figure 8D) Head H = 200 feet.

The net force acting downward on the plan area over the sloped part of the leaf bottom is:

(1)	Gate opening, %	10	30	40	50	60	80
(2)	$\frac{\text{Head diff}}{H_T}$ %	23.3	47.1	52.9	52.9	51.4	25.1
(3)	(2) x H_T = head diff (feet, H_2O)	42.6	94.2	105.0	105.8	102.8	50.2
(4)	(3) x 62.4 = Press diff (psf)	2,658	5,878	6,522	6,602	6,414	3,132
(5)	(4) x area = downpull, lb (Area = 6.0 x 1.75 = 10.50)	27,909	61,719	68,796	69,321	67,347	32,886

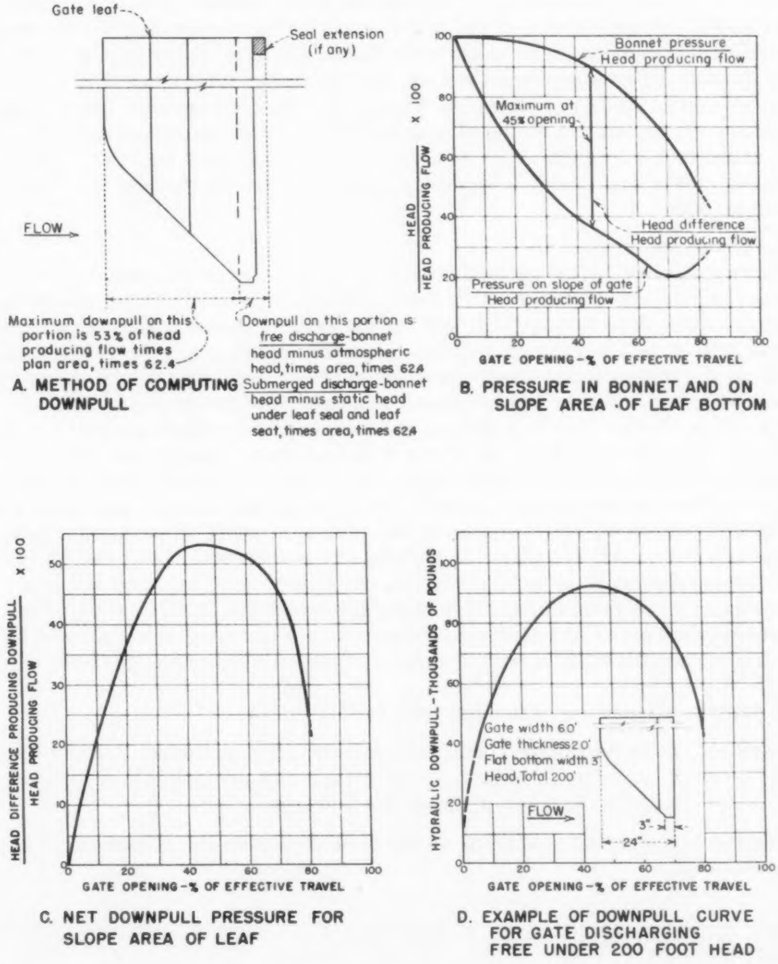


FIGURE 8 - PRESSURES IN BONNET AND ON LEAF, AND TYPICAL DOWNPULL CURVE FOR SLIDE GATE

DATA FROM AIR MODEL

The net force acting downward on the plan area of the flat bottom and the slot projections (no seal extension on standard gate) is:

(6)	Bonnet Pres % / H_T	99.5	96.0	92.2	86.3	78.0	50.1
(7)	(6) x H_T = Bonnet Pres = Head diff (ft, H_2O)	199.0	192.0	184.4	172.6	156.0	100.2
(8)	(7) x 62.4 = Pres diff (psf)	12,417	11,981	11,507	10,770	9,734	6,252
(9)	(8) x Area = Downpull; A = (0.25 x 6.0) + 2(0.5 x 0.5) = 2.00	24,834	23,962	23,014	21,540	19,468	12,504
(10)	(5) + (9) = Total downpull lb	52,743	85,681	91,810	90,861	86,815	45,390

(H_T = Total head upstream of gate.)

If it were assumed that the full total head acted over the entire sectional area of the leaf, the calculated downpull would be 200 x 62.4 x (10.5 + 2.0) = 156,000 pounds. The maximum downpull of 91,810 pounds obtained with the above test information is 59 percent of this value. This percentage will vary somewhat depending upon the width of the flat bottom of the leaf, the location of the leaf projections, and whether or not seal extensions are used.

Submerged Conditions

If the tailwater elevation rises to submerge the gate or fill the downstream conduit, and the headwater elevation remains unchanged, the head differential across the gate, and hence the rate of flow, decreases. In spite of these changes, and provided that the Reynold's Number remains reasonably high, the flow pattern approaching and passing along the face and sloping portion of the leaf remains the same. And because the flow pattern is the same, the difference between the upstream conduit total head and the head at any point on these areas of the leaf, divided by the head differential across the gate, remains the same for any gate opening. Expressed mathematically, $\frac{H_T - h_x}{H_T - h_2} = K$, and the downpull force on the sloped portion of the leaf will be the same as for the free discharge condition with the same differential head. In the case where a gate can be placed more deeply or less deeply below the tailwater elevation, the effect of submergence change is merely to equally raise or lower the bonnet and leaf slope pressures. The head differential across the gate would remain constant, and so would the downpull forces (assuming no cavitation at the lower submergences).

When the discharge is into a pool, the head producing flow may be taken as the difference between the total head upstream from the gate and the pool depth downstream. When the discharge is into a filled conduit, the head producing flow may be taken as the difference between the total head upstream and the static head a distance of about 3 conduit heights downstream.

For submerged flow where conditions would permit vapor pressure under the flat on the leaf, the method of calculation would be the same as that in the free discharge example except that the downpull head differential in Item 7 would become the total bonnet pressure plus the numerical value of the vapor pressure, about 30 feet of water. This produces the greatest downpull possible, but it is unlikely that vapor pressure would ever occur over this whole area. Thus, this extreme downpull condition will probably never be reached. In the more likely cases where conduit failures or unexpectedly low tailwater will not be factors, and where the submergence will always be sufficient to hold the pressures on the leaf flat and on the seal above vapor pressure, the net downpull would be determined from the difference between the bonnet pressure and these surface pressures. Such surface pressures may be difficult to predict exactly, and would probably have to be determined experimentally if precise results were needed. But for ordinary work, the pressures can be obtained from the non-dimensional data presented in Figure 7.

SUMMARY

Air models are reliable tools for determining flow phenomena and pressure distribution in fluid flow passages. As such, they are readily adaptable to studies of downpull forces on gates, particularly where the gates operate submerged. They may also be used for downpull studies when the gates discharge freely, provided that care is taken in analyzing the flow conditions and resulting pressure distributions on the various portions of the gate leaves. Air models are less expensive to construct and test than hydraulic models, and they are accurate, convenient, and easy to work with.

The two gate types discussed in this paper are typical of installations being made throughout the country. The data are presented in a form usable for any sized gate, provided that the top and bottom of the leaves are generally geometrically similar to the test gates. The maximum downpull on the tested fixed-wheel gate occurred during submerged operation at about a 3-inch opening. The downpull rapidly decreased as the opening increased. On the slide gate, the maximum downpull force occurred at about a 45 percent gate opening. This calculation assumed that the total head remained constant at the gate regardless of gate opening. In the usual case, friction in the conduit ahead of the gate will cause a loss of head as the flow rate increases. The maximum downpull would be less than in the example, and would occur at a smaller gate opening. Bonnet pressures play a particularly important part in the case of the slide gate, and in designs where leakage from the bonnet is permitted or encouraged to decrease the bonnet pressures, downpull will be materially reduced. Of course, this drainage or leakage may create other complications, and cannot be regarded as an automatic solution to the downpull problem.

ACKNOWLEDGMENT

The studies discussed in this paper were conducted in the Hydraulic Laboratory of the Bureau of Reclamation, and the fixed-wheel gate studies were made by Donald Colgate, Hydraulic Engineer.

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APPLICATION OF SNOW HYDROLOGY TO THE COLUMBIA BASIN

Oliver A. Johnson,[1] M. ASCE and Peter B. Boyer[2]

SYNOPSIS

Observations of the weather elements that cause the melting of a snow pack were made for several years in three laboratory areas located in different weather regimes of the high mountains of the western United States, and the data obtained were analyzed and correlated. Starting with the interrelationships between the weather elements that produce snowmelt at a point, methods and procedures were found for the rational analysis of snowmelt in a relatively small basin. The application will be enlarged to cover large parts of the Columbia Basin as the network of stations covering the basin is enlarged and additional climatological phenomena are measured.

INTRODUCTION

The Columbia River is principally a snow-fed stream. Its average annual runoff ranks very high among the streams of the United States, averaging about 180,000,000 acre-feet. The basin is located just off the Pacific Ocean in the belt of the heavily moisture-laden prevailing westerly winds. In winter the Aleutian lows attract maritime air masses which deposit their moisture as they ascend the high mountain slopes on the west side of the Cascades. The eastern slopes of the Cascades and the plains of the Columbia Plateau lie within the "rain shadow" of the Cascades, and annual precipitation in this region may be as little as eight inches. Additional uplift of the air masses by interior mountain ranges and the Rocky Mountains causes increased precipitation in the headwater regions, in amounts ranging to more than 80 inches per year.

Note: Discussion open until June 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1904 is part of the copyrighted Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. HY 1, January, 1959.

1. Hydr. Engr., North Pacific Div., Corps of Engrs., U. S. Dept. of the Army, Portland, Ore.
2. Hydr. Engr., Portland Dist., Corps of Engrs., U. S. Dept. of the Army, Portland, Ore.

The Columbia rises in Columbia Lake at elevation 2652 feet m.s.l. as compared to elevation 1670 for Lake Itasca, the source of the Mississippi. The mean elevation of the Columbia drainage east of the Cascades is about 4500 feet m.s.l. The rugged topography is important because of the large vertical distance through which the water descends on its way back to the Pacific, and because of the possibilities for economical construction of large dams afforded thereby. In January 1957, 25 million kilowatts of hydro capacity were available in the United States while that in Oregon, Washington, Idaho and that part of Montana west of the Continental Divide (comprising about 9 percent of the U. S. area) totalled more than 7 million kilowatts or 28 percent of the total.

During the design of a storage project on the Columbia or one of its tributaries, snow hydrology must be considered in the computations required in the selection of the project design flood. This is the largest flood that must be controlled if a satisfactory flood control plan is to be achieved. For that part of the Columbia and its tributaries lying to the east of the Cascade Range, the melting of the snow pack creates the severest design conditions. In the Willamette and other coastal streams the most intense floods are caused by severe rain-on-snow events occurring in the period November through February.

Power projects and flood control storage are entirely compatible in the storage projects above Bonneville Dam. Power draft through the fall and early winter will provide ample space for storage of floodwaters from moderate rainstorms that have occurred infrequently in December, and continued draft will provide additional space before the onset of the snowmelt season.

In the storage projects above Bonneville Dam there are monetary benefits to be derived by releasing stored water for power production at site and at downstream plants, and there are monetary benefits derived from flood damage prevented by such storage, both locally and in the vulnerable reach of the Columbia lying downstream from Bonneville Dam.

In the operation of the Columbia River projects, the principal objective is to provide storage space for a possible snowmelt flood and still have full reservoirs at the beginning of the power draft period. To achieve this objective, the water content of the snow on the ground is measured at frequent intervals throughout the accumulation and depletion period. With the current observations of precipitation and temperature and assuming normal weather for the remaining period, satisfactory runoff volume forecasts are made. These forecasts are increasingly more accurate as the season progresses because the future precipitation becomes progressively less important. Large deviations from normal precipitation during the melt period in May and June can cause the runoff volume to differ considerably from the forecasted event, but operating parameters can be devised that will afford satisfactory regulation of the projects.

There are other snow-fed streams that are of great importance to the United States, such as the Colorado, Sacramento, and Rio Grande, to name a few on which the Corps of Engineers and Bureau of Reclamation have built many reservoirs. A knowledge of the rates of accumulation and depletion of snow are of utmost importance for intelligent and efficient operation of these reservoirs. The Weather Bureau, Soil Conservation Service and Geological Survey also have a great interest because they are the principal agencies charged with the collection, processing and dissemination of hydrologic information. All are endeavoring to improve their forecast of the available basin snow water, of its rate of release and of losses. Reliable forecasts by

relatively rapid means is mandatory for both economical design and operation.

The need for detailed, accurate knowledge of the characteristic and behavior of snow was recognized by several of the Federal agencies and after some discussion, a program known as the "Cooperative Snow Investigations" was agreed upon and supported by the Weather Bureau and the Corps of Engineers in 1945. Limited assistance was furnished by the Geological Survey, Bureau of Reclamation and Forest Service. Direct participation by the Weather Bureau ended in 1952, and thereafter the basic research was continued by the Corps of Engineers under the name of "Snow Investigations." Throughout its life the program was pointed toward the solution of hydrologic problems pertinent to mountain regions of western United States. Three laboratory areas were heavily instrumented and studied intensively for periods of five to eight years in order to provide the basic data for determination of the hydrologic factors involved. These laboratory areas were designated Central Sierra (near Soda Springs, California), Upper Columbia (near Marias Pass, Montana), and Willamette Basin (on Blue River, in Oregon).

Beginning in 1954 a part of the group effort on snow investigations was directed toward compiling the results achieved by these studies in the form of a summary report that would serve as a reference volume for hydrologists. The work was concluded in June 1956 with a volume titled "Snow Hydrology"(1) containing some 400 pages of text and 72 plates. The principal features that apply to the Columbia River Basin are presented in this paper.

Point and Areal Relationships

The field of snow hydrology is concerned with the evaluation of various components of the hydrologic cycle. It draws upon the sciences of meteorology, hydraulics, thermodynamics, geology, soil mechanics and plant physiology. The development of the applied snow hydrology begins with observations and measurements, at a point, of factors affecting each component. This is followed by an analysis, leading to the establishment of mathematical relationships. Prior to basin wide application of the point values, the relative magnitudes of the components are determined, and emphasis is placed on the more important elements. Having determined point values of the components of the hydrologic cycle, the next problem is to modify the equations to fit the local climate and topography peculiar to an individual project basin. During the course of the snow investigations it was discovered that experimental data from areas as large as the laboratories (4 to 20 sq. mi.) could be fitted to point relationships only with difficulty, and near the end of the program experimental data from impervious areas measured in the hundreds of square feet were obtained to assist in evaluating the coefficients needed to determine point melt.

Having determined point values, the next step consists of utilizing these values to determine amounts and distribution over a basin area, with respect to differences in environment, as well as to time. This involves procedures which are much less exact than those for point evaluations. It is impractical to attempt a point-by-point analysis in basin application; rather, it is necessary to deal with basin averages in major subdivisions of geography or environment, and also to deal with averages in time. This concept leads to the use of point measurements as indexes to represent basin values. Since the

use of indexes involves the theory of sampling errors in measurement, statistical procedures may be employed to evaluate the reliability of estimates and also the weights assigned to individual factors. Intelligent selection of indexes requires an understanding of the heat processes and the relative effects of topography, forest and sky cover on the heat supply to the snow pack at a point.

Water-Balance in Areas of Snow Accumulation

When reliable hydrologic data are available, but the records are too short to justify a statistical approach, an accounting method, called water-balance, is employed in forecasting the stream runoff during a specified period. In addition, a knowledge of the water-balance for a given area is necessary in order to select appropriate forecasting parameters and to interpret their reliability. The water-balance technique may be used as a forecasting procedure by quantatively balancing runoff against precipitation, change in snowpack water equivalent, and losses. A knowledge of the water balance is also necessary for rate-of-flow forecasting and for design flood computations.

The water-balance method is formulated by the equation:

$$Q_g = W_1 + P - W_2 - L \tag{1}$$

in which P is the net basin precipitation (after correcting for gage catch deficiency and interception loss); W_1 and W_2 are the basin snowpack water equivalents at the beginning and end of the period; and L is the loss consisting of evapotranspiration (L_{et}) and soil moisture change (L_s), that is,

$$L = L_{et} + L_s \tag{2}$$

in which L_s is positive when water is added to the soil to reduce its moisture deficiency and negative when moisture is being evaporated from the soil; Q_g is the generated runoff volume during the period considered, equal to the observed runoff corrected for recession flow. Generally the purpose of a runoff forecast is to determine in advance the volume of water (Q) expected to enter the reservoir between two given dates. The forecasted Q_g is therefore modified to reflect this desired volume as follows:

$$Q = Q_g + Q_1 - Q_2 \tag{3}$$

whence equation (1) reduces to

$$Q = W_1 + P - W_2 - L + Q_1 - Q_2 \tag{4}$$

in which Q_1 and Q_2 are the runoff volumes under the initial and terminal recession curves for the period considered.

Snowmelt Relationships

Heat is required to melt the snow. At present we know enough about the principles of heat exchange between the snow and its environment to formulate general snowmelt equations based upon detailed studies for each of the three laboratory areas. Furthermore, these equations can be expressed in terms of the ordinarily available meteorological elements which properly represent

the heat processes. The level of our confidence is not as high in the application of the point snowmelt principles to basin-wide melt, since only a small number of basin-wide melt analyses have been completed. Each basin presents a separate snowmelt problem and has a unique solution. The principal problem is to integrate the effect of topography on the climate near the snow. This involves (1) the separation of the project basin into areas of homogeneous heat supply under cloudless, cloudy, and rainy sky conditions, and (2) determination of the basin constants for each area—constants which will integrate the influence of topography on solar radiation, on long wave radiation, and on convective-condensation heat exchange.

The processes responsible for the supply or less of heat are:

Absorbed solar radiation (H_{rs})

Net longwave radiation exchange between the snowpack and its environment (H_{rl})

Convective heat transfer (sensible heat) from the air (H_c)

Latent heat of vaporization released by condensate (H_e)

Conduction of heat from underlying ground (H_g)

Heat content of rain water (H_p)

These processes are influenced by meteorologic, geographic and time factors which must be considered and related to the ordinarily observed data. The general equation for snowmelt at a point is:

$$M = \frac{\Sigma H}{203B} = \frac{\Sigma H}{203(0.97)} = 0.0051 \Sigma H \tag{5}$$

in which M is in inches, ΣH is the algebraic sum of heat energy by short wave radiation, long wave radiation, convection, condensation, conduction and precipitation; 203 (= 80 x 2.54) is the number of calories required to melt one inch of water equivalent of ice at 0° C over a unit area; B is the thermal quality which is equal to the ratio of heat required to melt a unit weight of the snow to that of ice. $B > 1$ when the snow temperature is less than 0°C and $B = 1$ when the snow is at 0°C and dry. But a melting snow, after drainage, contains liquid water which requires no heat to melt. Therefore, its thermal quality, after drainage, varies somewhat with density. An average value of $B = 0.97$ is satisfactory, as shown in equation (5).

The analysis of observations and measurements of radiation, temperature, humidity, condensation (or evaporation) and melt at a point resulted in the following equations, where snowmelt, M, for each heat component is in inches per day.

Shortwave Radiation Melt

$$M_{rs} = 0.0051\ (1-a)\ I_i \tag{6}$$

where a is the albedo (reflectivity) of the snow surface, expressed as a decimal fraction and

I_i is the incident shortwave radiation in langleys (Cal. / sq. cm.).

Longwave Radiation Melt

For clear skies

$$M_{rl} = .022\ (T_a - 32) - 0.84 \tag{7}$$

where T_a is the air temperature at the 10-foot level in ^{o}F.

For cloudy skies (low clouds)

$$M_{rl} = 0.029\ (T_c - 32) \tag{8}$$

where T_c is the temperature of the cloud base in ^{o}F.

For forested area

$$M_{rl} = 0.029\ (T_a - 32) \tag{9}$$

Convection Melt (theoretical)

$$M_c = 0.00629\ (p/p_o)\ (Z_aZ_b)^{-1/6}(T_a - T_s)\ V_b \tag{10}$$

where p/p_o is the ratio of the atmospheric pressure at the site to standard sea-level pressure.

Z_a is the height in feet above the snow surface of the air temperature measurement.

Z_b is the height in feet above the snow surface of the wind speed measurement.

T_a is the air temperature in ^{o}F.

T_s is the snow surface temperature.

V_b is the wind speed in miles per hour.

Condensation Melt (theoretical)

$$M_e = 0.054\ (Z_a\ Z_b)^{-1/6}(e_a - e_s)\ V_b \tag{11}$$

where Z_a is the height in feet above the snow surface of the vapor pressure measurement.

Z_b and V_b are as defined above.

e_a is the vapor pressure of the air in millibars.

e_s is the saturated vapor pressure of the snow surface.

Combined Convection and Condensation Melt (simplified)

$$M_{ce} = [0.0084\ v]\ [0.22(T_a - 32) + 0.78\ (T_d - 32)] \tag{12}$$

where v is the wind speed at the 50-foot level in miles per hour.

T_a is the air temperature at the 10-foot level in ^{o}F.

T_d is the dew point temperature at the 10-foot level in ^{o}F.

Conduction of Heat from Ground

Melt from this source is negligbly small, so that direct evaluation is unnecessary. Average value is approximately 0.02 inches/day.

Rain Melt

$$M_r = 0.007\ (T_r - 32)\ P_r \tag{13}$$

where T_r is the temperature of rainwater in $^{\circ}F$ (usually assumed to be that of the air temperature at the 10-foot level).

P_r is the rainfall in inches.

A practical method of establishing a reliable clear-weather melt equation, in terms of the available heat indexes, for a ripe snow in a given basin is by means of statistics. The influence of topography and forest cover on each heat process will then be integrated in derived coefficients that will also include the period of melt and the constants of proportionality between units of measure. The proposed form of such a linear regression equation is:

$$Q_g = k'(1-a)I_i + AT_a\ k_{ce}\ (DT_a + CT_d - T_b)v + D \tag{14}$$

where Q_g is the generated runoff. The term, $k'(1-a)I_i$, is the solar radiation melt in which I_i is the solar radiation incident on the horizontal surface of the snow area, a is the albedo of the snow, and k' is the basin constant integrating the effect of topography and forest cover on solar radiation. The term CT_a defines the long wave radiation melt with C as its derived coefficient. The term, $k_{ce}(BT_a + CT_d - T_b)$ v, is the convection-condensation melt, in which k_{ce} is a constant defining the degree of basin exposure to wind; T_a and T_d are the air and dew point temperatures; B, C, and T_b are derived constants, and v is the wind speed. D is the regression constant. Ground melt is negligibly small and it will be partly reflected in the other terms.

The melt at the snow surface for the generation period is:

$$M = Q_g + L \tag{15}$$

in which L is the loss of water by evapotranspiration and deep percolation. Note that the proposed statistical analysis assumes L as a function of T, T_d and I. Evapotranspiration is a function of these elements; and deep percolation, unless definitely established, is assumed to be negligible.

Examination of data taken on cloudy and cloudless days indicates that an adjustment factor for average cloudiness should be used in computing long wave radiation melt. On the days of complete cloud cover and on rainy days, it is easier to develop surface melt equations by a rational approach.

Melt equations peculiar to each sub-basin may be derived statistically by direct correlation of appropriate meteorologic factors to daily generated runoff amounts as explained above. In many cases, it may not be feasible to derive basin snowmelt equations in this manner because of lack of appropriate data. General snowmelt equations were therefore developed from laboratory data, which may be used to evaluate basin snowmelt on the basis of the local physical characteristics and the meteorologic factors affecting snowmelt. The physical characteristics of a basin affecting heat supply may be expressed in

terms of coefficients of average forest cover, exposure to wind, and exposure to solar radiation. The form of the equation is determined by the amount of forest cover, since it has a marked effect upon the relative importance of both wind and solar radiation in estimating snowmelt. For rain-free periods, the meteorological parameters include free air and dewpoint temperature, wind speed, solar radiation and cloud cover. During periods of active melt, the only variable of the snowpack condition is the albedo and the snow cover. For estimating snowmelt during rain, values of air temperature, wind speed, and rainfall are required.

The components of the clear weather melt (in inches per day) were each modified by the appropriate basin factor to account for slope and orientation (k'), forest cover shading (F), exposure to wind (k), and for cloud cover (N), resulting in the following equations:

Shortwave radiation melt, M_{rs}:

$$M_{rs} = 0.0051\ k'(1-F)(1-a)\ I_i \tag{16}$$

Longwave radiation melt, M_{rl}:

$$M_{rl} = (1-N)(1-F)\ [0.022\ (T_a-32) - 0.84] + [N + F(1-N)]\ 0.029(T_a-32) \tag{17}$$

Convection-Condensation melt, M_{ce}:

$$M_{ce} = 0.0084\ kV_{50}\ [0.22\ (T_a-32) + 0.78(T_d-32)] \tag{18}$$

Ground melt: a constant 0.02 in. per day.

In the above equations I_i is the daily short wave radiation (in langleys) reaching a horizontal surface at the latitude and altitude of the snow area; T_a and T_d are the daily average air temperature and humidity at 10 feet above the snow; and V_{50} is the daily average wind speed (in mph) 50 feet above the snow. This study was followed by a statistical analysis of the clear weather snowmelt in each of the three laboratories, using estimated basin average temperature, wind, k, and F values. The analysis resulted in the following typical snowmelt formulas, in which k, k' and F are estimated from a careful examination of the topography of the snow area.

Heavily forested area (like Willamette Basin Snow Laboratory) having more than 80 percent forest cover:

$$M = 0.074\ (0.53T'_a + 0.47\ T'_d) \tag{19}$$

Forested area (like Skyland Creek, Upper Columbia Snow Laboratory) having 60 to 80 percent forest cover:

$$M = k\ \ (0.0084\ v)(0.22T'_a + 0.78\ T'_d) + 0.029T'_a \tag{20}$$

Partly forested area (like Central Sierra Snow Laboratory) having 25 to 60 percent forest cover:

$$M = k'(1-F)(0.0040I_i)(1-a) + k(0.0084v)(0.22T'_a + 0.78T'_d) + F(0.029T'_a) \tag{21}$$

Open area (less than 25 percent forest cover):

$$M = k'(0.0051 I_i)(1-a) + (1-N)(0.022\,T_a' - 0.84) + N(0.029 T_c') + k(0.0084\,v)(0.22 T_a' + 0.78 T_d') \qquad (22)$$

where:

M is the snowmelt in inches per day over the snow covered area.

T_a' is the difference in air temperature measured at 10 feet above the snow and the melting snow surface temperature, in ^{o}F. The forecasted temperature of the air at a station may be extrapolated to the snow level by the lapse rate. The temperature of the melting snow is 32 ^{o}F.

T_d' is the difference between the dewpoint temperature at 10 feet above the snow and the melting snow surface, in ^{o}F. It is approximated from upper air or surface humidity forecast. The dewpoint of the melting snow is 32^{o}F.

T_c' is the difference between the cloud base and the melting snow surface temperatures in ^{o}F. It is estimated from the upper air temperature or by a lapse rate from the surface temperature.

v is an estimated representative wind speed at 50 feet above the snow, in miles per hour. Wind stations in the proximity may serve as a guide.

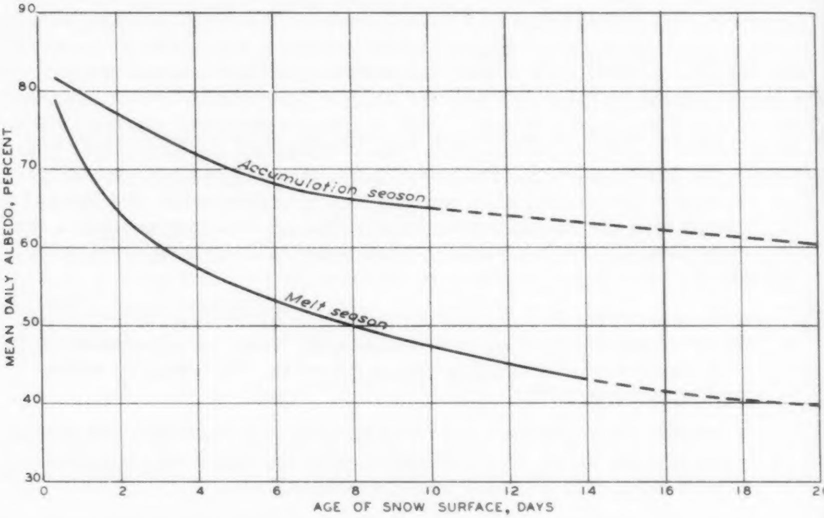
I_i is the estimated short wave radiation on a horizontal surface at the elevation and latitude of the snow area in langleys per day. It may be estimated from observed station values near the area or indirectly from duration of sunshine, cloud cover, or diurnal temperature variation.

a is the estimated albedo of the snow surface. Values of a can be estimated from figure 1. A melting snow has a smaller albedo than the non-melting snow; the albedo of new dry snow may be as high as 0.85 and falls very rapidly to lower values with time, following a decay-type curve. The accumulation of residual impurities, as snow melts, tends to decrease the albedo and in forests it may be as low as 0.35.

F is an estimated average basin cover, effective in shading the area from solar radiation, estimated as a decimal fraction. The estimate of F involves the consideration of the type, height and density of the forest cover.

k' is a factor intended to correct I_i for an average slope angle and orientation of the snow covered area. The effective short wave radiation decreases as a sloped surface is oriented from south toward north, and as the slope angle is farther away from normal to the sun's rays. In effect $k' = I/I_i$, the ratio of the insolation on the true snow surface to that on its horizontal projection. Figure 2 shows I/I_i values for the entire year for a southerly and for a northerly oriented unit area of 25^{o} slopes in 46^{o} 30 N Latitude at 5200 feet above mean sea level.

k is an average corrective factor for the degree of exposure of the snow area to wind. This factor has a maximum value at 1.0, but it may be as low as 0.3 for a forested area.



VARIATION IN ALBEDO WITH TIME

FIGURE 1

N is the estimated cloud cover, expressed as a decimal fraction.

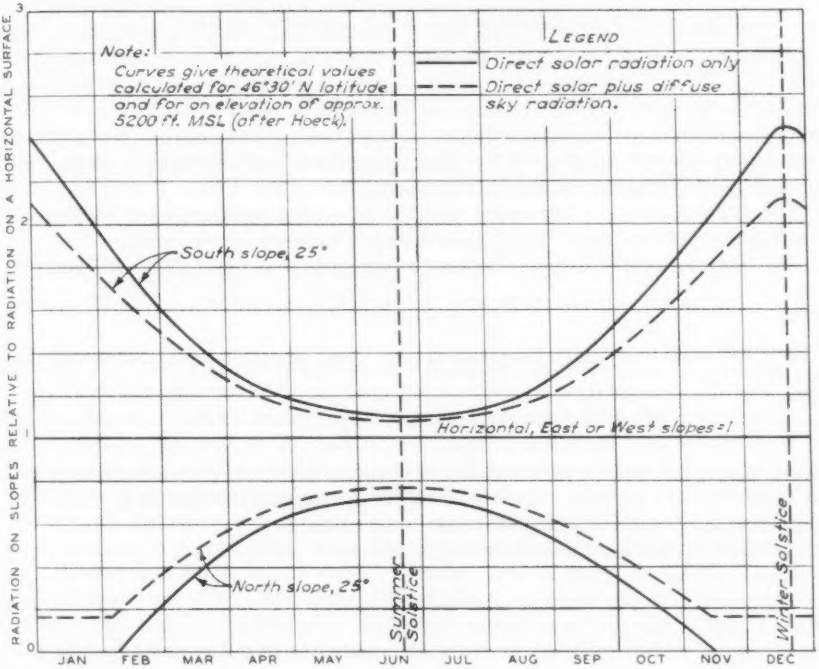
Attention is called to equation 19 in which k', I_i, v, and k are missing. In a heavily forested area, the effect of wind is dampened to a near constant value and insolation is a small portion of the total heat supply, approximately equal to evapotranspiration. The heat energy, effecting the generated snowmelt runoff, essentially includes convection-condensation and long wave radiation. These components are satisfactorily defined by a linear function of the air temperature (T) and the dewpoint (T_d), within the range experienced during the melt period. A combination of both the rational and statistical approach resulted in the coefficients shown in equation (19) for the generated daily snowmelt runoff.

Equation (20), for the forested area, has been derived by a rational approach, assuming insolation melt plus ground melt equal to evapotranspiration; and therefore, the generated daily snowmelt runoff (M_g) equals the convection-condensation melt plus the longwave melt.

When equations (19) and (20) are used, "water excess" for routing is obtained by subtracting only the infiltration water which is not available to direct runoff within the hydrograph period. Equations (21) and (22) are derived by rational approach, considering point surface melt. The snowmelt M is the water input at the surface and includes the evapotranspiration.

The snowmelt during rain was derived from clear weather melt equations. Since $T_d = T_a$ is a permissible substitution during a rain-on-snow event, the daily convection-condensation melt from equation (18) for an open to partly forested area is:

$$M'_{ce} = 0.0084\ k\ V_{50}\ (T_a - 32) \qquad (23)$$



DAILY SOLAR RADIATION ON CLEAR DAYS ON NORTH AND SOUTH SLOPES OF 25-DEGREE GRADIENT RELATIVE TO RADIATION ON A HORIZONTAL SURFACE

FIGURE 2

For a forested to heavily forested area, where wind may be considered a constant, a statistical treatment gave

$$M'_{ce} = 0.045\ (T_a - 32) \tag{24}$$

Assuming the temperature of the cloud base, forest foliage, and the air above the snow are the same, the long wave radiation melt would be

$$M'_{rl} = 0.029\ (T_a - 32) \tag{25}$$

As a result of rain heat, assuming the air temperature = the rain temperature, the rain melt

$$M'_r = 0.007\ (T_a - 32)\ P_r \tag{26}$$

where P_r is the daily rainfall.

The short wave melt varies with the intensity of the rain. But it is very small and it is considered satisfactory to express it as a constant or as a

function of the forest cover factor F, like

$$M'_{rs} = 0.05 \text{ for forested to heavily forested areas.}$$

$$M'_{rs} = 0.10 \text{ for lightly forested and open areas, or}$$

$$M'_{rs} = 0.05\ (2-F) \tag{27}$$

Ground melt is considered negligible, if any, during a time when the ground heat supply may be used up by the moving surface and subsurface water. Melt by rain impact (kinetic energy) is insignificant.

The components are combined, and the following equations are written to represent the daily snowmelt values during a rain-on-snow event:

For open to lightly forested areas, using equations (23), (25), (26) and (27)

$$M' = (T_a - 32)(0.0084\ kV_{50} + 0.007\ P_r + 0.029) + 0.05\ (2-F) \tag{28}$$

For forested to heavily forested areas, using equations (24), (25), (26) and (27)

$$M' = (T_a - 32)(0.074 + 0.007\ P_r) + 0.05\ (2-F) \tag{29}$$

Note that for a given basin k and F are constants. When they are determined, the equations are further simplified, allowing the construction of a simple table or nomograph for zonal or basin melt values.

The snowmelt over the entire basin (M_b) is:

$$M_b = M \frac{A_s}{A_b} \tag{30}$$

where A_s/A_b is the ratio of the snow covered area to the area of the entire basin.

Effect of Snow Pack Conditions on Runoff

Water passing through a snowpack may freeze if the snow is "cold" and dry; it may be retained against gravity as hygroscopic and capillary water; it may be retarded or detained by ice dams, by choking, and by suspension in a form of slush; or it may continue in well-defined channels. In a mountainous area of snow cover all of these phenomena may occur simultaneously during winter and early spring.

As in other mediums, water will move through the snowpack in a path of least resistance. The rate of movement and the storage of water in snow depends on the topography, the temperature, depth, density and the crystaline structure of the snowpack. Regardless of its path, before water can penetrate through a layer of the snowpack, the path in that layer is brought to $0^{o}C$ by the heat of fusion of the freezing water; each snow particle in the path is coated with water, and the capillary spaces filled. Only the excess continues downward. Upon reaching an impervious ice plane, the water flows over it until a weakness in the ice plane allows part or all to pass through to the next snow layer below. Cells of cold snow between ice planes may still remain in the pack when water reaches the ground level. However, continued rain and melt will eventually disintegrate all ice planes, change the crystaline structure and render the entire pack isothermal and saturated to capacity. At this stage, the water will move downward almost vertically, and the snowpack will

yield all input to runoff with little delay. Water freezing within the pack and water held against gravity (not including any possible slush water in level areas) constitute the retention storage or the initial loss to direct runoff. Such water is retrieved only as the snow melts. The magnitude can be computed, and the time delay determined from the input intensity.

The cold content is the heat required to raise the temperature of a "cold" snowpack to 0°C. Its water equivalent is the amount of water that would supply the required heat (80 calories per gram) by freezing within the pack. Taking the specific heat of snow at 0.5, the cold content (h) in calories is:

$$h = 1.27\, W_o T_s; \qquad (31)$$

and its water equivalent (W_c) in inches is:

$$W_c = 0.00625\, W_o T_s \qquad (32)$$

when W_o is the initial snowpack water equivalent in inches and T_s is the average snow temperature in degrees below 0°C. If a known amount of water of 0°C entered a "cold" snowpack of uniform texture and density, the generated runoff appearing at the bottom of the pack would be less than the inflow by an amount equal to the water equivalent of the cold content (W_c) plus the hygroscopic and capillary capacity of the snow.

Like soil, the snowpack is capable of supporting hygroscopic and capillary water in amount varying with the mass of the pack, which is a function of size, spacing and shape of the snow crystals. The density, as an integrating factor of the structure of the snowpack, appears to be an index of the amount of such liquid water. Laboratory experiments with melting snowpack of 0.43 to 0.56 g/cc, after complete drainage, showed retention of liquid water amounts, held against gravity, from 2 percent to 7 percent by weight. No studies are known to show the order of magnitude of the free water holding capacity of the winter snowpack which has generally an average density less than 0.43 g/cc. The light winter snowpack with small grains has greater surface area to be wetted but larger voids which may not be capable of supporting capillary water. Light dry snow, of course, can not remain light after drainage. When rain falls on it, the snow immediately settles and reaches a density near 0.25 g/cc after which the settlement rate is relatively slow. Since there is a lack of information on the free water holding capacity of the winter snowpack of relatively light density, the lower limit (3 percent) of the experimental results is considered satisfactory. In general then the liquid water holding capacity of the snowpack against gravity (after drainage) is stated as

$$S_f = f''_p\, (W_o + W_c) \qquad (33)$$

in which S_f is the water stored and f''_p is the liquid-water-holding capacity of the snowpack in percent of W. In the winter, a value of $f''_p = 0.03$ should prove satisfactory, and in the spring for the daily thawing of the night crust a value of $f''_p = 0.04$ or 0.05 is recommended.

The magnitude of the retention storage (S_r), as an initial loss, may now be represented as the sum of the water equivalent of the cold content and the liquid water holding capacity of the snowpack, or

$$S_r = 0.00625\, W_o T_s + f''_p\, (W_o + W_c) \qquad (34)$$

In a natural ripe snowpack in which the planes are disintegrated and the pack completely primed to yield runoff, the water travels vertically at about 30 to 60 inches per hour depending inversely on the density; the denser the snow, the slower the velocity. But the time of travel (t_1) in a ripe snow is

$$t_1 = D/V \tag{35}$$

in which D is the depth and V is the velocity of water in the snowpack. The velocity of water in a ripe snowpack is relatively large compared to the velocity in a cold and dry winter snow. In a cold snowpack the time of travel (t) depends on the intensity of the inflow or rainfall (r) plus melt (m), the retention storage capacity, and depth of the pack. That is,

$$t = \frac{S_r}{r + m} + \frac{D}{V} \tag{36}$$

Where S_r is the amount shown in equation (34), and t is the total time elapsed from the beginning of rain to the time of appearance of appreciable runoff at the bottom of the pack. The relative magnitude of the retention storage, (S_r) and time delay (t) are shown by the following examples:

(1) Given: a cold winter snowpack at 7,000 ft. elevation where the snow depth is 50 inches with an average density of 0.30 g/cc and temperature of -5°C; rainfall and melt of 2.40 inches is forecast for the 24-hour period at an average rate of 0.1 inch per hour. Find W_o, the snowpack water equivalent, S_r, and t.

$$W_o = 50\ (0.30) = 15$$

Using $f''_p = 0.03$ and V = 40 ins. per hour, $S_r = 0.00625\ (15)\ 5 + 0.03\ [(15) + 0.00625\ (15)\ 5] = 0.93$ ins., leaving 1.47 inches for runoff which will begin to appear under the pack at this site after $t = \frac{0.93}{0.10} + \frac{50}{40} = 10.5$ hours from the beginning of the rain.

At time of appearance of water under the pack, approximately 0.13 in. ($= \frac{50}{40}\ (0.10)$) is in the snowpack, moving downward. If this was the first rain the snowpack had experienced, $t > 10.5$ because of its detention storage capacity in the form of several inches of slush at the bottom of the pack and perhaps on several ice planes.

(2) Given: At an open site, on a clear spring night, the top 6 inches of a snowpack cooled progressively to an average temperature of -3°C by 0500 hours (sunrise); this cold layer and the remainder of the pack had a density of 0.50 g/cc; below this crust the pack was drained of water except for its retention storage. From solar radiation and temperature forecast, it is estimated that the melting will begin at 0530 hours, its intensity varying with time, between 0530 and 1100 hours, as follows: $m = 0.01\ t^2$ in/hr. Find S_r. Using p = 0.05, the night crust will require $S_r = 0.00625\ (6)\ (0.5) + 0.04\ (6)\ (0.5) = 0.18$ in. It can be seen that ripening the snow crust each day requires an appreciable amount of the daily melt.

Note that the average temperature and the water equivalent of the snowpack must be known or estimated, before the retention storage and time delay can be computed.

Application of Principles within Columbia Basin

The principles and procedures developed as a result of the Snow Investigations have assisted in solving problems in snow hydrology in those areas of the mountainous western states investigated by the Corps of Engineers. Sudden melting of a moderate snow cover over the Northern Plains areas can cause excessive floods, and some of the actions and reactions in these cases are not yet completely solved. In the Columbia Basin, use of these principles has assisted in obtaining seasonal volume runoff forecast procedures, standard project floods, spillway design floods, and snowmelt relationships and indexes.

The Detroit project, located on a tributary to the Willamette River, is a multi-purpose project in which runoff from snowmelt floods is detained and used for the generation of power. The North Santiam above Detroit Reservoir is located on the west slope of the Cascade Mountains about 60 miles southeast of Portland, Oregon, and has a drainage area of 438 square miles. Elevations range from 1200 to 10,495 feet above mean sea level, the mean elevation being 3718 feet. A large percentage of the area is comprised of valleys and ridges with steep slopes, and a heavy stand of coniferous trees covers most of the area. Soil cover is relatively thin, but there is considerable duff and litter under the heavy forest canopy. The climate is characterized by wet, moderately cold winters and dry warm summers. Normal annual precipitation over the area is estimated to be 82 inches, ranging from below 70 near the Detroit project to well over 100 at the crest, where snow accumulates to great depths. About 90 percent of the precipitation occurs during the October through May period.

The water balance method, discussed earlier in this paper, was used for forecasting the volume of runoff from the basin above this project, and the procedure used is considered representative of a method adaptable to an average project basin. Several of the steps taken in this procedure deviate from the procedure derived for a laboratory area. No estimate was made for losses by interception, since these are small compared to the total rainfall, which in turn is not accurately known, having been estimated by summing the generated runoff and the calculated evapotranspiration loss. Likewise, no estimate was made of gage-catch deficiency due to the wind.

Precipitation, snowfall, streamflow, air temperature, and snowpack water equivalent data are available for varying periods. The only adequate temperature record available is that at Detroit, necessitating the use of lapse rates to obtain estimated temperatures at higher levels. Precipitation data are available for five stations, one of which was begun in 1909. Water equivalent is measured at four snow courses having records since 1941. Because of regulation during the construction of Detroit Dam, the streamflow record subsequent to 1951 was not considered usable for this investigation, and the period of record suitable for study extends from the beginning of snow-course records (1941) through 1951.

The snowpack remaining on the basin as of August 31 each year is negligible, and the reservoir is filled after the threat of a rain-type winter flood has largely passed, so a reliable forecast procedure for the runoff available from February 1 through August 31 was needed. Forecasts covering the periods March-August and April-August were also needed. Equation (4) is used for all periods. The basic equation used for all periods is as follows:

$$Q = P + (W_1 - W_2) + (Q_1 - Q_2) - L \quad (37)$$

in which Q is the available runoff during the period from the date of the forecast to August 31. P is precipitation, W_1 and W_2 are the initial and final snowpack water equivalents respectively, Q_1 and Q_2 are the runoff volumes under the initial and terminal recession curves, and L is loss, all values being estimated as inches of depth over the drainage area. The final snowpack water equivalent, W_2 is equal to zero in this case. The snowpack water equivalent for the date of the forecast (W_1) is obtained from a plot of the water-equivalent at each of three key stations, at known elevations, from which an indication of the water equivalent for each 10 percent of the basin area is read. The average snowpack water equivalent is obtained by application of a factor derived from a study of the relationship of precipitation, loss and runoff during the 11-year period of study. Q_1 is the recession volume corresponding to the discharge on the date of the forecast and Q_2 is the recession volume corresponding to the discharge on the last day of the forecast period. Losses, L, were computed by Thornthwaite's method for each year, based on temperatures at Detroit. This method relates losses to climate, and estimates losses by an empirical formula using only air temperature as a variable, with adjustments for precipitation experienced and for the latitude of the area considered, which indicates the length of the day.

Precipitation during the forecast period must be accounted for in use of the forecasting procedure, since spring precipitation constitutes a major portion of the water supply during this period. Values of precipitation may be assumed on the basis of normal or critical conditions during the spring period; or someday it may be possible to utilize long-range precipitation forecasts.

On February 1, the initial snowpack water equivalent W_1 amounts to only about 25 percent of the total of W_1 plus P; this forecast is greatly dependent upon future climatological events and is generally to be considered a preliminary estimate. On April 1, the value of W_1 is about 60 percent of the total of W_1 plus P, and the forecast of this date is reliable enough to be of major assistance in the operation and refilling of the Detroit reservoir.

The following is the February 1-August 31, 1958 runoff forecast of the water balance method:

February 1, 1958 (11-year average - 13.2 ins.),	W_1	= + 14.6 in.
February 1-August 31 normal precipitation,	P	= + 30.6 in.
February 1-August 31 normal losses,	L	= - 8.8 in.
Recession runoff volume for observed February 1 discharge (6227 cfs)	Q_1	= + 9.2 in.
Recession runoff volume for normal August 31 discharge (633 cfs)	Q_2	= - 5.1 in.
February 1-August 31 runoff forecast,	Q	= 40.5 in.

Cougar Dam, now under construction on a tributary to the Willamette River, is to be a multi-purpose project storing water for flood control and power production. A standard project flood is derived for projects similar to this, in order to determine the amount of flood control storage that should be provided if economically feasible. This flood is one which would be

exceeded only upon rare occasions caused by a rainstorm falling upon a snow covered basin. It requires the combination of rather severe antecedent conditions and a temperature sequence that would cause some of the snow to melt and contribute to the flood. The largest flood recorded at the dam site occurred in 1953, and this rain-on-snow event was considered to precede the standard project flood.

The 210 square-mile drainage area above the Cougar dam site, elevation 1532 feet mean sea level was divided into five elevation bands of about 1000 feet each. Analysis of snow surveys and the climate of the region indicated that the basin could reasonably be expected to have a 100 percent snow cover with 8.2 inches of water content at the dam site and an increase of 7.5 inches of water content for each of the five bands. This is considerably more than the normal and could accumulate only under temperatures less than normal. It was conservatively assumed that the basin snow cover was at 32°F and required only hygroscopic and capillary water of about 2 percent by weight before yielding runoff, amounting to an initial loss of 0.55 inches, part of which was regained as the snow melted. Melt due to short wave radiation and ground heat is insignificant during a rainstorm and was omitted. The snowmelt was computed by zones, using a melt rate of 0.08 inches per degree-day above 32°F applied to appropriate air temperatures for each zone. The melt rate conforms to that previously determined at the Willamette Basin Snow Laboratory for the condition of rain-on-snow in heavily forested areas. Snowmelt from rain itself was added separately. The hydrograph of this flood would have a peak discharge of 40,000 c.f.s. or 190 c.f.s. per square mile, and a total volume of 194,000 acre-feet. Storage of 154,000 acre-feet of runoff would be required to control this flood, and a usable capacity of 155,000 acre-feet is being provided.

Some of the variables for clear-sky snowmelt are neglected in this computation. The heavy forest on this basin markedly reduces the wind speed variation over the snow surface. Therefore, a wind variable is unnecessary, and evaluation of convective melt can be accomplished by use of air temperatures alone. With intense rainfall the air would be saturated, so that the dewpoint temperature would be equal to the air temperature. For this reason, condensation melt may also be evaluated from an air temperature function alone. Rainfall over the basin during the most intense 6-hour period was estimated to be 3.26 inches, and snow melt during this same period was 0.72 inches. Total rainfall for the design storm amounted to 10.75 inches, and snowmelt totalled 3.64 inches. The temperature at the 3900 foot level was assumed to reach a maximum of 49°F.

Several spillway design floods have been computed for projects in the Columbia River Basin using the procedures and relationships that have been developed as a result of the Snow Investigations. Such a flood was developed for the Libby project, on the Kootenai River in Canada, and just recently for projects on the Salmon River in Idaho. These determinations require a considerable amount of detailed work, and they will not be described here.

A very important application of snow hydrology in the Columbia Basin is evolving in the day-by-day use of data gathered by electronic means which must be interpreted and used in producing a forecast of flood flows and gage heights in the ensuing short-term period. Reservoir regulation for flood control and flood fighting operations at local critical areas and in the reach of leveed river downstream from Bonneville Dam will be assisted thereby. Because a forecast must be completed within a short time after the raw data

become available to be of use, the hand or desk machine computations entering into them have been restricted to a relatively few. With the advent of a high speed digital computer, and a routine written for the Columbia Basin, forecasts will be made for a large number of small tributary basins, based upon antecedent weather and forecasts of temperature and precipitation to be expected in the next several days. The same routine will route the flows from the small tributaries through lakes and channel storage and will furnish plotted hydrographs at key points. Such a routine requires the derivation of indexes and coefficients pertaining to each of the small tributary basins.

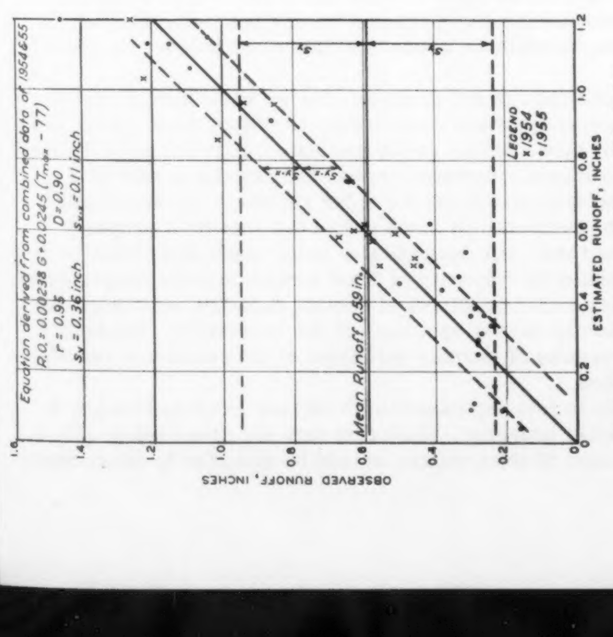
One of the tributary basins selected for analysis was that of the Boise River above the Twin Springs gaging station, where the 1954 and 1955 spring snowmelt hydrographs were used. The Boise River above Twin Springs, Idaho drains an area of 830 square miles of the Sawtooth Mountains. The area has deep valleys, steep slopes and narrow sharp-topped ridges, ranging in elevation from about 3000 to 9000 feet above mean sea level. The stream system and instrumentation are shown in Fig. 3. Forest cover consists principally of conifers which are estimated to have a shading effect on 30 percent of the basin area. Snowmelts for each day of the periods were computed by the thermal budget index and the flows resulting from this released water plus the precipitation for that day were routed to the gaging station for comparison with the observed hydrograph. This comparison for the May-June 1955 period is also shown in Fig. 3. The index derived for the snowmelt over this basin is:

$$M = 0.0267\, T_{max} + 0.00227G - 2.00 \tag{38}$$

where M is the daily snowmelt in inches over the basin, T_{max} is the daily maximum temperature, at Idaho City, in degrees F, and G is a radiation parameter, in langleys (equal to absorbed shortwave radiation plus estimated longwave loss.)

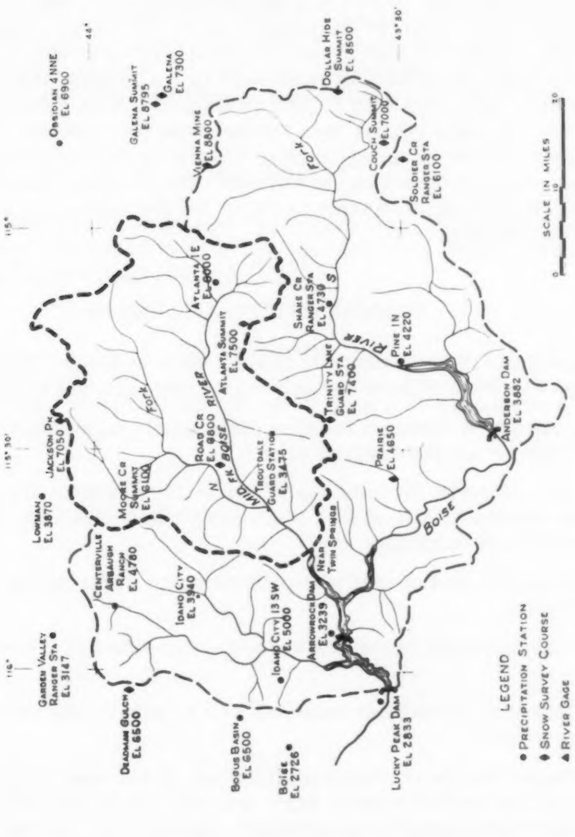
It will be noted that one of the terms in the above equation requires that a forecast of the radiation to be experienced on ensuing days must be available. Radiation is measured daily at Boise, Idaho and these data were used in deriving the equation, but large parts of the Columbia Basin do not have such measurements available. This refinement may be used sometime in the future, when more reliable forecasts of the temperature and precipitation and other weather factors are possible and when data on radiation become more widespread. For the present, a simple maximum daily temperature index of snowmelt is being used, which provides fairly reliable estimates of snowmelt runoff for day-to-day forecasts of streamflow.

Another application of indexes to the predication of flood volumes resulted from the Snow Investigations. The method is based primarily upon the relationship between winter runoff of low-elevation drainage basins in western Washington and Oregon, and the spring snowmelt runoff of the Columbia River at The Dalles. Indexes of winter temperature and spring precipitation are included in the forecast procedures as secondary parameters. The use of low-elevation winter flow as an index is confined to regions where the low-elevation and the high-elevation areas have a common source of moisture. Such a situation exists in the region composed of the Columbia Basin and western Washington and Oregon, the entire region being well centered in the belt of prevailing westerlies. Moisture is carried in a generally eastward direction from the Pacific Ocean, and the amount deposited is largely



R.O. = Daily generated snowmelt runoff in inches depth over snow-covered area
G = Daily net allwave radiation absorbed by snow in open, langleys
T_{max} = Daily maximum temperature, Boise, °F
r = Coefficient of correlation
D = Coefficient of determination
S_y = Standard deviation of observed runoff, inches
S_{yx} = Standard error of estimated runoff, inches

OBSERVED VERSUS ESTIMATED RUNOFF



HYDROMETEOROLOGICAL STATIONS

FIGURE 3

dependent upon the rate of flow and the preciptable moisture content of the air, reflected by both winter streamflow from low elevation areas and the accumulation of snow at high areas. Several tests indicated that a correlation of winter flow from two streams each in Washington and Oregon would yield results as accurate with regard to historical data as those which use precipitation and snow course data for the principal indexes. This method does not produce any information as to the distribution of the snow pack over the smaller interior basins of the Columbia River above The Dalles, Oregon, but it does produce an acceptable forecast for the basin as a whole quite early in the accumulation period.

SUMMARY AND CONCLUSIONS

The significant developments in applied snow hydrology which have resulted from the work of Snow Investigations research and analysis are:

A point snowmelt equation based on heat supply from short and longwave radiation, convection and condensation.

A general snowmelt formula applicable to any drainage area when overall coefficients characteristic of that basin are evaluated for the degree of exposure to wind and radiation.

Relative magnitudes of melt components under different sky and forest cover conditions.

Numerical appraisal of the storage and delay effect of the snow pack on runoff.

Runoff forecast procedures based on water-balance and on a coastal winter-flow index.

In addition to the above direct applications, numerous investigations were performed which were of a more basic nature. These deal with the theoretical aspects of snow physics, precipitation, snow cover, and soil moisture measurements, forest effects on snow accumulation and evapotranspiration loss, heat exchange between the snowpack and its environment, the distribution of the snowpack in mountainous areas, and the water balance in areas of snow accumulation.

Our knowledge of snowmelt is not complete, but we know enough now to supplement the degree-day approach by including humidity, wind speed, and radiation values even if they must be partly estimated, particularly for clear weather melt in open and partly forested areas. Only during a rain in a dense forest, where the wind speed and the solar radiation are small and nearly constant, can the temperature alone define the snowmelt properly.

Direct application of these principles is now being made in estimating design floods for determining the capacity of flood control storage projects and river channels, and the related problems of conduit capacity, revetment design, levee heights, and day-to-day operation of the reservoirs. Seasonal operation is based on results of monthly forecasts of the remaining runoff by the water balance method.

The network of stations is being expanded to include more hydrologic information and better areal coverage. Radiation data are observed at only a few stations and this class of observations should be extended to cover more

area. The snow fields in the high mountains are uninhabited, and observations of climatoligical events in this large and important area are difficult to obtain but steady advances are being made by use of radio-reporting facilities. The distribution of snow cover during the melt period is also being observed on a systematic basis by means of aerial snow reconnaissance.

ACKNOWLEDGMENT

Much of the material for this paper was taken from the Summary Report on Snow Hydrology, published in June 1956 by the North Pacific Division, Corps of Engineers, Portland, Oregon.

REFERENCE

1. "Snow Hydrology," North Pacific Division, Corps of Engineers, U. S. Army, Portland, Oregon, dated 30 June 1956.

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FLOW THROUGH CIRCULAR WEIRS[a]

Closure by J. C. Stevens

J. C. STEVENS,[1] M. ASCE.—The author is gratified that six engineers well versed in mathematics and hydraulics took time to send in discussions of this paper.

The experiments by Mr. Manabar are very interesting even though his pipe was small. The sharp edge of the inlet of a pipe on a steep slope is not the same as a sharp-edged circular weir through which free flow obtains. When partly full the inflow was partially submerged and this is correctly assigned by him as the reason his average flow coefficient of .52 was less than the author's average of .59. His Fig. 2 indicates that the stage-discharge relation is virtually a straight line for higher values of H/D. This is in conformity with the graphs for the laboratory flows listed in Appendix A.

The head on circular weirs need not be limited at all to $H<D$ for the weir may be surcharged; also it may be submerged and it is often necessary to calculate the flow under these conditions.

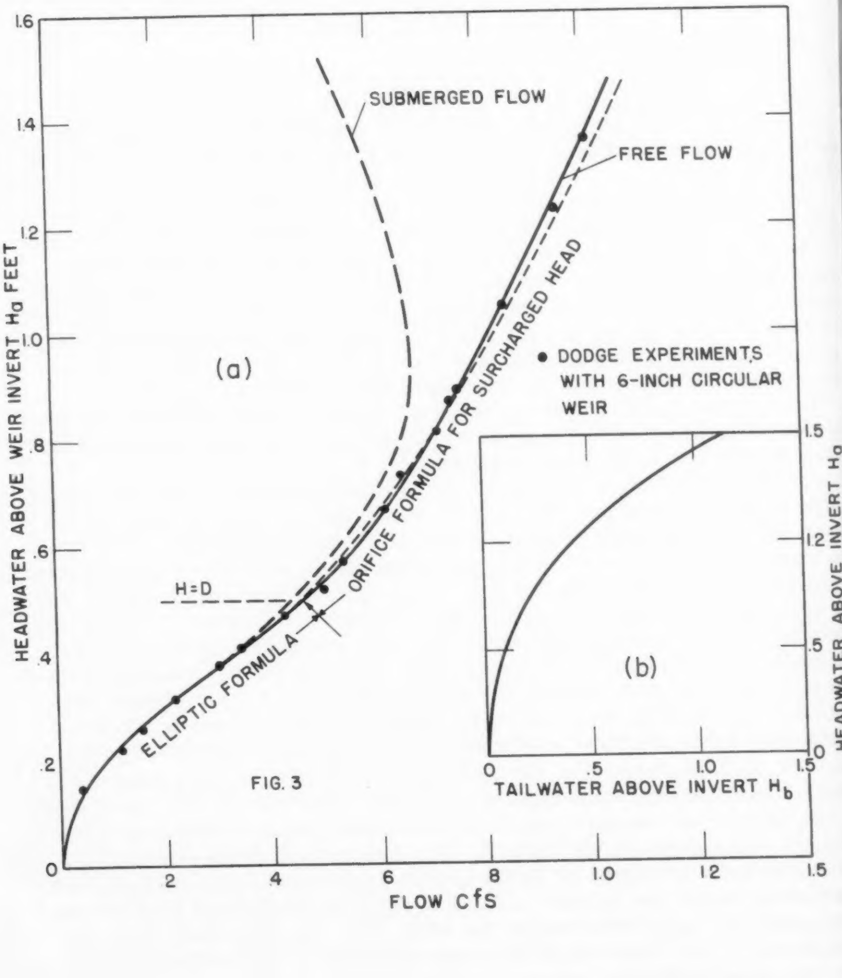
For surcharged circular weirs there are two conditions to be met. In the range $H\leq D$ flow is readily calculated from the elliptic formula with the aid of the tables in the author's paper. When $H>D$ the weir becomes an orifice for which the common orifice formula is applicable, the head being measured from the center. Fig. 3 shows the calibration of a 6-inch sharp-edged circular weir by E. R. Dodge to a head of $H=D$ (Appendix A). However, his thesis(3) gives the flow for surcharged heads. The weir was in the outlet end of and concentric with a 15-inch pipe in which the approach velocity was negligible. The average flow coefficient of .607 given in Appendix A for this weir was used with the Equation (3) to define the solid line Fig. 3 (a) for $H>D$. For surcharged heads the same coefficient was used in the orifice formula viz $Q = .607\ A\sqrt{2gh}$ where $A = .197$ sq ft and h is the head on the center of the circular weir-orifice. The small circles represent Dodge's laboratory flows.

Mr. McPherson has given the effect of surcharging the 1-foot weir from Omori's experiments in his Table B. Fig. 4 is a graph of the flow through the Omori weir calculated by Eq (3) up to $H/D = 1.00$ using the average flow coefficients of the last column in Table A. For the surcharged head the corresponding average coefficient for the weir, .596, was also used. Note the smoothness of the transition of flow weir to orifice in both examples which are well substantiated by laboratory experiments.

For submerged flow the Villemonte formula (4) may be used. For a weir of any kind subject to submersion in a stable channel observations will soon

a. Proc. Paper 1455, December, 1957, by J. C. Stevens.

1. Cons. Engr., Stevens & Thompson Engrs., and Leupold and Stevens Instruments, Inc., Portland, Ore.



1.6
1.4
1.2
1.0
.8
.6
.4
.2
0
HEADWATER ABOVE WEIR INVERT H_a FEET
SUBMERGED FLOW
FREE FLOW
ORIFICE FORMULA FOR SURCHARGED HEAD
DODGE EXPERIMENTS WITH 6-INCH CIRCULAR WEIR
(a)
H=D
ELLIPTIC FORMULA
2
4
6
8
1.0
1.2
1.5
FLOW cfs
(b)
0
.5
1.0
1.5
TAILWATER ABOVE INVERT H_b
HEADWATER ABOVE INVERT H_a

FIG. 3

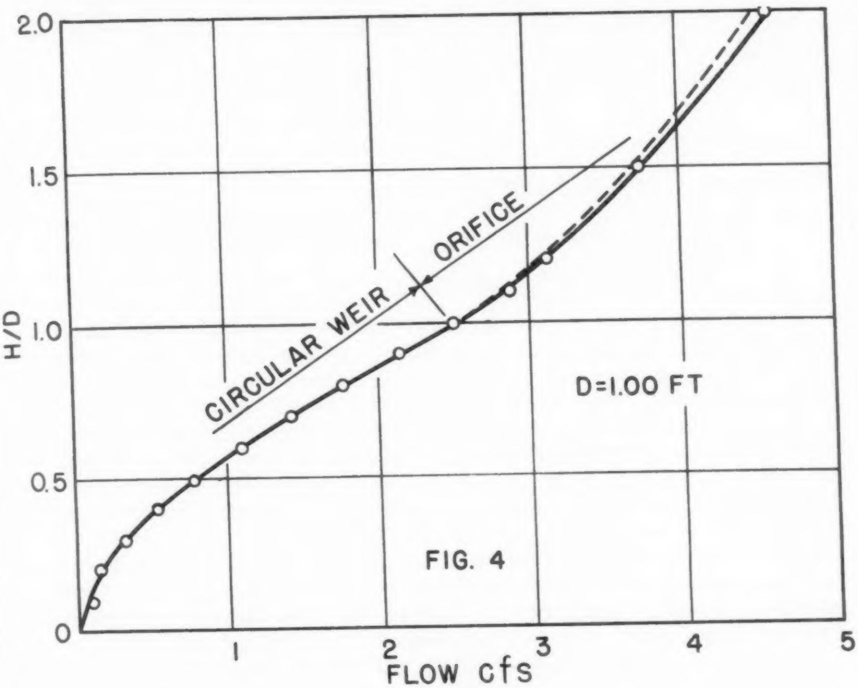

2.0
1.5
1.0
0.5
0
H/D
CIRCULAR WEIR
ORIFICE
D=1.00 FT
1
2
3
4
5
FLOW cfs

FIG. 4

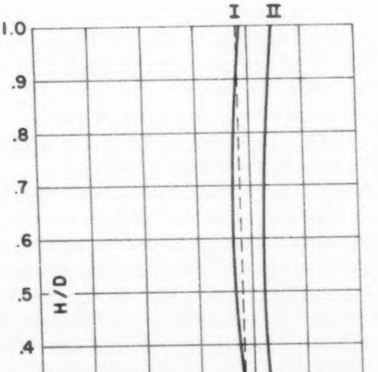

I
II
1.0
.9
.8
.7
.6
.5
.4
.3
.2
.1
0
H/D
DISCHARGE COEFFICIENT
.50
.60
.70

FIG. 5

define a relation between headwater and tailwater. To illustrate, Fig. 3 (b) shows such a relationship (similar to one found for the outlet of a grit channel with a proportional sharp edged weir at its outlet.)

The general form of the Villemonte Formula is

$$\frac{Q}{Q_a} = k\left(1 - \frac{Q_b}{Q_a}\right)^m \tag{4}$$

Where Q is the observed flow -cfs.

Q_a, the flow corresponding to the headwater (above)

Q_b, the flow corresponding to the tailwater (below)

k, a constant

m, an exponent to be determined

Both Q_a and Q_b are taken from the free flow curve of Fig. 3 (a). For most sharp-edged weirs k is practically 1.0 and m has the average value of .385 found by Dr. Villemonte's laboratory experiments. A nomogram is readily constructed for the Villemonte formula for any set of stable conditions.(15)

In Fig. 3 the dashed line shows the effect of submerging the weir-orifice. This well illustrates the versatility of the circular weir-orifice. What other type of open channel measuring device lends itself to reasonable accuracy throughout a head range from zero up to a hundred or more feet? Even the Venturi tube is not accurate for low rates of flow.

The author is grateful indeed for the errata in Mr. Blaisdell's discussion. As one grows older his facility for errors increases markedly, especially with the slide rule. The flow experiments have been checked and revised tables of Appendix A, where errors were found, are appended hereto.

The table of Appendix C is revised in its entirety in order to eliminate the minor errors in the 4th decimal place for which Mr. Moody takes the author to task and quite justifiably so. Any table should be correct to the last decimal. Mr. Moody's admirable table of values of Δ to 4 decimals with consistent 2nd is a step in the right direction but interpolation may be necessary. The author believes that k^2 should be carried to 3 decimal places.

Mr. Kolupaila's table gives the theoretical flow directly for a 1-foot diameter weir. The argument x has been substituted for k^2 in the author's Eq (3). The theoretical flow through a circular weir of any other diameter D is obtained by multiplying the tabular value by $D^{5/2}$.

In answer to Mr. Blaisdell and doubtless others who noticed the lack of information regarding the approach conditions for the weir data in Appendix A, references for the experiments now include this information.

The author knew nothing about the Byrd and Freidman Handbook of Elliptic Integrals for Engineers and Physicists, Moody's Ref (1). He has since purchased a copy and was quite non-plussed to find included 2 errata pages of fine print — about a hundred corrections. In the sweet-bye-and-bye perhaps there'll be no errors - "Tis a consumation devoutly to be wished."

In weir computation the orthodox method followed throughout the entire history of hydraulics has been to compute the theoretical flows, then compare them with observed discharges and correct the computed flows by a factor which might be called a coefficient of ignorance.

For all sharp edged weirs that coefficient is some measure of the effect of the sharp edges distorting the stream lines as water passes the weir. Mr. Blaisdell's use of his formula (a) for circular weirs is unorthodox but certainly permissible. To be utterly capricious one might even measure H from

any arbitrary datum, reduce or increase a by a constant or a percentage, even use the value of g from Mars or Venus and obtain "goofy" discharges (Q_{nuts}) computed from (A). Parallel observed discharges would then determine a series of coefficients that would define a "goofy" curve but from which the true discharges Q could doubtless be found.

The average coefficient C = .365 (Fig. A), he admits gets top-heavy beyond H/D = .8. Dividing the scattered values of his c by the author's corresponding scattered values of C (Fig. 2) he obtains the tantalizing curve of Fig. B which the author at first mistook for an improved set of discharge coefficients applicable to circular weirs. However, Mr. Blaisdell, by letter, disclaims any such intention and states "the values in my Fig. B are a measure of the incorrectness of my approach, part of which I suggested might result from my neglect of what Professor Mavis calls 'shape factor.' I worked it out and compared my results with yours as a matter of interest. . . I thought others might also be interested." (See his Ref. 3).

The data on discharge coefficients presented by Messrs. McPherson and Kolupaila are shown in Fig. 5. Curve I indicates the average coefficients of the 4 experimenters given in Table A of Mr. McPherson's discussion viz Ormori, Staus, Staus & Sander, and Greve.

Curve II gives Dr. Kolupaila's coefficient formula which he slightly modified from that of Staus as follows:

$$C_d = 0.570 + \frac{1}{100x} + 0.041x$$

where x = H/D. The Staus formula used 0.555 in place of 0.570.

The author's constant coefficient of .59 fits Curve I fairly well for moderate to high values of H/D. The deviations are -9% at H/D = .10, -5% at .20, and practically zero above .30.

Mr. Moody has offered a new and valuable approach to orifice flow by his formulas (3) and (4). In order to avoid confusion it would be preferable to use $(D/H)^2$ rather than k^4. For theoretical flow through vertical circular orifices he proposes

$$Q_t = \frac{16\,\Delta}{15\pi(D/H)^2}\,A\sqrt{2gh} \tag{4}$$

which readily reduces to

$$Q_t = 2.14\Delta H^{5/2} \tag{21}$$

Where Δ is the elliptic function given in Appendix C using D/H for the argument instead of k^2 = H/D, while H is measured from the invert of the orifice as in the weir instead of from its center. Obviously D/H < 1.00.

Fig. 3 shows by a dotted line the results of using (21) for the flow through Dodge's 6-inch orifice, using the average weir coefficient of 607. (Appendix A). Fig. 4 shows the corresponding flow for Omori's 1-foot orifice in Table B using the average coefficient for Omori's weir experiments of .583 given in the first column of Table A. It appears therefore that the average discharge coefficient found for the circular weir may be used as the average coefficient for the same weir when it is surcharged becoming an orifice as is indicated by the four examples herein given. More laboratory experiments on surcharged circular weirs would be fruitful.

ERRATA

Pages 1455-5&6 - Substitute the following for the corresponding numbered references:

1. Cone, V. M. Circular Weirs. Journal of Agricultural Research USDA March 6, 1916. The approach flume was 20 feet long 10 feet wide and 6 feet deep. For the 2-foot weir the flume bottom was 4-1/2 feet below the center of the weir.

2. Greve, F. V., Flow of Water through Circular Parabolic and Triangular Vertical Notch Weirs, Engineering Bulletin, Research Series No. 40. Purdue University, Lafayette, Indiana. This bulletin also contains a bibliography with summaries but no mention of European experiments. The approach channel was 16 feet long and 5 feet wide. The depth below the weir is not given but it scales 2-1/2 feet below the invert of the largest weir.

3. Dodge, Eldon R. A Thesis Submitted for a Masters Degree, University of Wisconsin, Madison, Wisconsin. 1935. (Not published). The sharp-edged circular weirs reported herein were concentrically in the end of two sizes of tile sewer pipe as follows:

Weir Nominal D	Pipe I D	Weir Nominal D	Pipe I D
2-1/2 -inch	8-inch	4-inch	15-inch
4 -inch	8-inch	6-inch	15-inch

5. Thijsse, J. Th. Discharge Meters Director Hydraulic Laboratory (Waterloopkundig Laboratorium) Delft, The Netherlands. Three sizes of flumes were used. Weirs were not rated beyond H/D = .75. For the 2.36-foot (72 cm) weir the nominal dimensions of the flume were, width 3 feet, depth below invert of weir 1.0 ft length of 9 ft. There is strong evidence that the increase in the coefficient for all 3 sizes is due to velocity of approach. However these weirs were calibrated in place.

For the 1.77 foot (54 cm) weir the flume dimensions were width 2.2 feet, depth below invert of the weir .75 feet, length 7.0 feet.

For the 1.181 foot (36 cm) weir the flume length was 4.6 feet, width 1.5 feet, depth below invert 0.5 feet.

Page 6 add the following references:

14. Submerged-weir Discharge Studies by James R. Villemonte, Engineering News-Record Dec. 25, 1947.

15. Proportional Weirs for Sedimentation Tanks, by J. C. Stevens, Proc. Paper 1015, June 1956.

Appendix A - Substitute the following tables for those of the corresponding Author and diameter.

E. R. Dodge

D	H	H/D	Q_t	Q	C
.211	.084	.398	.018	.012	.666
	.100	.474	.025	.016	.640
	.108	.512	.029	.019	.655
	.132	.626	.042	.026	.619
	.143	.677	.048	.031	.646
	.150	.711	.052	.031	.596
	.152	.720	.053	.032	.604
	.169	.802	.063	.036	.572
	.171	.810	.064	.039	.619
	.177	.839	.068	.043	.633
	.189	.896	.075	.046	.614
	.193	.915	.077	.050	.649
	.202	.958	.083	.051	.614
	.211	1.000	.087	.055	.632
					.626

F. W. Greve

D	H	H/D	Q_t	Q	C
.50	.131	.262	.071	.043	.606
	.157	.314	.101	.060	.594
	.160	.320	.105	.061	.580
	.206	.412	.168	.098	.583
	.212	.424	.177	.104	.588
	.273	.546	.282	.164	.580
	.305	.610	.344	.200	.580
	.344	.688	.424	.244	.575
	.391	.782	.525	.304	.580
	.440	.880	.633	.366	.580
					.585

V. M. Cone

D	H	H/D	Q_t	Q	C
.50	.20	.40	.159	.10	.628
	.25	.50	.241	.13	.540
	.30	.60	.334	.19	.568
	.35	.70	.437	.25	.572
	.40	.80	.545	.30	.550
	.45	.90	.655	.38	.580
	.50	1.00	.758	.44	.580
					.574

E. R. Dodge

D	H	H/D	Q_t	Q	C
.336	.128	.380	.054	.034	.628
	.142	.423	.066	.040	.606
	.161	.478	.082	.050	.610
	.168	.500	.089	.055	.618
	.219	.652	.144	.084	.583
	.229	.682	.155	.097	.625
	.235	.700	.162	.096	.593
	.243	.723	.174	.101	.580
	.262	.780	.195	.114	.584
	.273	.812	.208	.122	.587
	.294	.875	.233	.136	.585
	.299	.888	.239	.139	.583
	.302	.900	.244	.151	.619
	.321	.955	.265	.156	.589
					.599

F. W. Greve

D	H	H/D	Q_t	Q	C
1.333	.280	.210	.540	.322	.597
	.322	.242	.709	.417	.588
	.415	.311	1.149	.675	.588
	.495	.372	1.611	.935	.580
	.577	.433	2.139	1.244	.580
	.582	.437	2.176	1.260	.580
	.703	.527	3.069	1.797	.586
	.814	.610	3.986	2.296	.575
	.967	.726	5.380	3.070	.570
	1.124	.844	6.873	3.900	.567
	1.146	.860	7.077	3.995	.565
	1.332	1.000	8.780	5.100	.580
					$\overline{.580}$

F. W. Greve

D	H	H/D	Q_t	Q	C
1.748	.159	.091	.204	.128	.627
	.207	.118	.345	.214	.620
	.254	.145	.520	.316	.608
	.321	.184	.825	.482	.597
	.388	.222	1.183	.910	.600
	.435	.249	1.476	.875	.593
	.487	.279	1.837	1.080	.589
	.528	.302	2.141	1.250	.584
	.588	.345	2.756	1.540	.558
	.647	.370	3.143	1.830	.582
	.703	.402	3.670	2.140	.583
	.750	.429	4.142	2.390	.577
	.801	.458	4.677	2.710	.579
	.853	.488	5.257	3.040	.578
	.907	.519	5.883	3.400	.579
					$\overline{.590}$

V. M. Cone

D	H	H/D	Q_t	Q	C
1.50	.20	.133	.298	.18	.604
	.25	.167	.467	.25	.535
	.30	.200	.660	.38	.575
	.35	.233	.886	.51	.575
	.40	.267	1.152	.69	.598
	.50	.333	1.758	1.03	.586
	.60	.400	2.480	1.45	.584
	.70	.467	3.308	1.90	.575
	.80	.533	4.211	2.44	.580
					$\overline{.579}$

D	H	H/D	Q_t	Q	C
2.495	.311	.125	.942	.573	.608
	.409	.164	1.607	.966	.601
	.453	.181	1.943	1.175	.604
	.490	.196	2.265	1.359	.600
	.526	.211	2.608	1.550	.594
	.563	.226	2.978	1.765	.593
	.585	.235	3.209	1.895	.590
	.620	.248	3.560	2.118	.594
	.665	.267	4.105	2.424	.590
	.699	.280	4.498	2.648	.589
	.723	.290	4.814	2.823	.588
					$\overline{.596}$

F. W. Greve

D	H	H/D	Q_t	Q	C
2.25	.280	.124	.715	.450	.630
	.351	.156	1.128	.690	.611
	.413	.183	1.534	.840	.548
	.525	.233	2.441	1.50	.614
	.627	.279	3.451	2.02	.585
					$\overline{.618}$

V. M. Cone

D	H	H/D	Q_t	Q	C
3.00	.30	.100	.953	.48	.504
	.40	.133	1.686	.81	.481
	.50	.167	2.640	1.22	.463
	.60	.200	3.730	1.72	.462
	.70	.233	5.006	2.30	.459
	.80	.267	6.506	2.95	.454
	.90	.300	8.142	3.67	.452
	1.00	.333	9.932	4.44	.447
					$\overline{.465}$

F. W. Greve

D	H	H/D	Q_t	Q	C
.75	.191	.254	.185	.107	.579
	.223	.297	.250	.148	.592
	.308	.411	.461	.263	.570
	.312	.416	.471	.268	.570
	.356	.475	.603	.345	.572
	.363	.484	.624	.356	.570
					.575

D	H	H/D	Q_t	Q	C
.833	.189	.227	.194	.112	.583
	.224	.269	.269	.158	.587
	.276	.332	.402	.231	.575
	.290	.348	.440	.254	.578
	.348	.418	.620	.358	.579
	.417	.502	.869	.495	.570
	.491	.590	1.162	.659	.575
	.495	.595	1.183	.671	.567
	.571	.685	1.508	.864	.573
	.654	.785	1.894	1.076	.569
	.661	.794	1.929	1.093	.567
	.669	.803	1.965	1.114	.568
	.752	.902	2.354	1.345	.571
	.755	.906	2.376	1.352	.569
	.833	1.000	2.750	1.570	.570
					.574

F. W. Greve

D	H	H/D	Q_t	Q	C
1.00	.205	.205	.251	.146	.580
	.254	.254	.379	.220	.580
	.307	.307	.546	.314	.576
	.364	.364	.754	.430	.577
	.430	.430	1.029	.594	.578
	.520	.520	1.461	.836	.572
	.526	.526	1.491	.850	.570
	.634	.634	2.079	1.190	.573
	.732	.732	2.659	1.520	.572
	.873	.873	3.530	2.000	.566
	1.000	1.000	4.280	2.520	.590
					.576

V. M. Cone

F. W. Greve

D	H	H/D	Q_t	Q	C
1.167	.253	.217	.413	.237	.575
	.316	.271	.633	.364	.575
	.390	.334	.944	.545	.575
	.492	.422	1.463	.835	.576
	.603	.517	2.128	.122	.578
	.708	.606	2.827	.160	.565
	.825	.708	3.701	.208	.561
	.947	.812	4.639	.261	.565
	1.166	1.000	6.300	.360	.572
					.571

Page 6, 3rd line before Acknowledgments substitute:

Jorissen, Andre, Le Déversoir Circulaireen Mince Poroi (A Study of the

Appendix A p 11 Heading of Table for D = 1.00 substitute:

V. M. Cone

Appendix A p 12 Heading of Table for D = 1.50 substitute:

V. M. Cone

Appendix B p 14 line 2 substitute:

$$Q = 1.262\ D^{5/2} \left[2(1 - k^2 + K^4)\ E - (2 - k^2)\ (1 - k^2)\ K \right]$$

Appendix C p 17 line 1 substitute

Values of $\Delta = 2(1 - k^2 + k^4)\ E - (2 - k^2)\ (1 - k^2)\ K$

Appendix D p 22 Equation (3) retype

$$y = H\ sn^2\ x = D\ k^2\ sn^2\ x \qquad (3)$$

Appendix D p. 21, line 17 substitute

$$F(\phi, k) = u = \int_0^{\phi} \frac{d\phi}{\sqrt{1 - k^2 \sin \phi}} \quad [k < 1] \quad [x = \sin \phi] = \int_0^x \frac{dx}{\sqrt{(1-x)^2 (1 - k^2 x^2)}} \qquad \text{DWIGHT 750}$$

Same, p. 22, 1st line substitute

$$E = \int_0^{\pi/2} \sqrt{1 - k^2 \sin^2 \phi}\ d\phi \qquad \text{DWIGHT 774.1}$$

Same, p. 23, 1st line Eq. 12 substitute

$$I = \int_0^K sn^2 x\ dx - (1 - k^2) \int_0^K sn^4 x\ dx + k^2 \int_0^K sn^6\ dx \qquad (12)$$

Same, P. 23, line 6 substitute

$$3 \int_0^K sn^2 x\ dx = 4(1 + k^2) \int_0^K sn^4 dx - 5k^2 \int_0^K sn^6 x\ dx + sn^3 x\ cn\ x\ dn\ x$$

Appendix D p 22 line 4

$$y = H\,sn^2\,x = D\,k^2 sn^2\,x \quad (3)$$

Same p22 12th line from bottom substitute:

$$= \left[4D^5 k^8 sn^4\,x\,cn^4\,x\,dn^4\,x\right]^{\frac{1}{2}} dx \quad (9)$$

Same p 22 4th line from bottom substitute:

$$cn^2 x = 1 - sn^2 x, \text{and } dn^2 x = 1 - k^2 sn^2 x$$

Same p 24 Eq (20) substitute

$$Q_t = \frac{4}{15}\sqrt{2g}\;D^{5/2}\left[2\,(1 - k^2 + k^4)E - (2 - k^2)\,(1 - k^2)\,K\right] \quad (20)$$

In Text:

Page 2 line 16 substitute:

Experiments by V. M. Cone[1]

Line 4 from bottom substitute:

weir flow was V. M. Cone who made his experiments in 1916 at Fort Collins.

Page 4 line 18 substitute:

Appendix (C). The following table gives Δ to 4 decimal places for values of k^2 to 3.

Page 5 line 4 substitute:

The experiments by V. M. Cone are all near the 59 per cent line except

k^2	0	1	2	3	4	5	6	7	8	9
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0001	0.0001	0.0002	0.0002
.01	0.0003	0.0004	0.0004	0.0005	0.0006	0.0007	0.0008	0.0008	0.0009	0.0011
.02	0.0012	0.0013	0.0014	0.0015	0.0017	0.0018	0.0020	0.0021	0.0023	0.0025
.03	0.0026	0.0028	0.0030	0.0032	0.0034	0.0036	0.0038	0.0040	0.0042	0.0044
.04	0.0047	0.0049	0.0051	0.0054	0.0056	0.0059	0.0062	0.0064	0.0067	0.0070
.05	0.0073	0.0076	0.0079	0.0082	0.0085	0.0088	0.0091	0.0094	0.0098	0.0101
.06	0.0104	0.0108	0.0111	0.0115	0.0119	0.0122	0.0126	0.0130	0.0134	0.0138
.07	0.0142	0.0146	0.0150	0.0154	0.0158	0.0163	0.0167	0.0171	0.0176	0.0180
.08	0.0185	0.0189	0.0194	0.0199	0.0203	0.0208	0.0213	0.0218	0.0223	0.0228
.09	0.0233	0.0238	0.0243	0.0249	0.0254	0.0259	0.0265	0.0270	0.0276	0.0281
.10	0.0287	0.0293	0.0298	0.0304	0.0310	0.0316	0.0322	0.0328	0.0334	0.0340
.11	0.0346	0.0353	0.0359	0.0365	0.0372	0.0378	0.0385	0.0391	0.0398	0.0404
.12	0.0411	0.0418	0.0425	0.0432	0.0439	0.0446	0.0453	0.0460	0.0467	0.0474
.13	0.0481	0.0489	0.0496	0.0503	0.0511	0.0518	0.0526	0.0533	0.0541	0.0549
.14	0.0557	0.0564	0.0572	0.0580	0.0588	0.0596	0.0604	0.0612	0.0621	0.0629
.15	0.0637	0.0646	0.0654	0.0662	0.0671	0.0679	0.0688	0.0697	0.0705	0.0714
.16	0.0723	0.0732	0.0741	0.0750	0.0759	0.0768	0.0777	0.0786	0.0795	0.0805
.17	0.0814	0.0823	0.0833	0.0842	0.0852	0.0861	0.0871	0.0881	0.0890	0.0900
.18	0.0910	0.0920	0.0930	0.0940	0.0950	0.0960	0.0970	0.0980	0.0990	0.1001
.19	0.1011	0.1022	0.1032	0.1042	0.1053	0.1064	0.1074	0.1085	0.1096	0.1106
.20	0.1117	0.1128	0.1139	0.1150	0.1161	0.1172	0.1183	0.1194	0.1206	0.1217
.21	0.1228	0.1240	0.1251	0.1263	0.1274	0.1286	0.1297	0.1309	0.1321	0.1332
.22	0.1344	0.1356	0.1368	0.1380	0.1392	0.1404	0.1416	0.1428	0.1440	0.1453
.23	0.1465	0.1477	0.1490	0.1502	0.1515	0.1527	0.1540	0.1552	0.1565	0.1578
.24	0.1590	0.1603	0.1616	0.1629	0.1642	0.1655	0.1668	0.1681	0.1694	0.1708
.25	0.1721	0.1734	0.1747	0.1761	0.1774	0.1788	0.1801	0.1815	0.1828	0.1842
.26	0.1856	0.1870	0.1883	0.1897	0.1911	0.1925	0.1939	0.1953	0.1967	0.1981
.27	0.1995	0.2010	0.2024	0.2038	0.2053	0.2067	0.2081	0.2096	0.2110	0.2125
.28	0.2140	0.2154	0.2169	0.2184	0.2199	0.2213	0.2228	0.2243	0.2258	0.2273
.29	0.2288	0.2303	0.2319	0.2334	0.2349	0.2364	0.2380	0.2395	0.2410	0.2426

k^2	0	1	2	3	4	5	6	7	8	9
.30	0.2441	0.2457	0.2473	0.2488	0.2504	0.2520	0.2535	0.2551	0.2567	0.2583
.31	0.2599	0.2615	0.2631	0.2647	0.2663	0.2679	0.2696	0.2712	0.2728	0.2745
.32	0.2761	0.2777	0.2794	0.2810	0.2827	0.2844	0.2860	0.2877	0.2894	0.2910
.33	0.2927	0.2944	0.2961	0.2978	0.2995	0.3012	0.3029	0.3046	0.3063	0.3080
.34	0.3098	0.3115	0.3132	0.3150	0.3167	0.3185	0.3202	0.3220	0.3237	0.3255
.35	0.3272	0.3290	0.3308	0.3326	0.3343	0.3361	0.3379	0.3397	0.3415	0.3433
.36	0.3451	0.3469	0.3487	0.3506	0.3524	0.3542	0.3560	0.3579	0.3597	0.3616
.37	0.3634	0.3653	0.3671	0.3690	0.3708	0.3727	0.3746	0.3765	0.3783	0.3802
.38	0.3821	0.3840	0.3859	0.3878	0.3897	0.3916	0.3935	0.3954	0.3973	0.3993
.39	0.4012	0.4031	0.4050	0.4070	0.4089	0.4109	0.4128	0.4148	0.4167	0.4187
.40	0.4207	0.4226	0.4246	0.4266	0.4286	0.4305	0.4325	0.4345	0.4365	0.4385
.41	0.4405	0.4425	0.4445	0.4465	0.4486	0.4506	0.4526	0.4546	0.4567	0.4587
.42	0.4607	0.4628	0.4648	0.4669	0.4689	0.4710	0.4731	0.4751	0.4772	0.4793
.43	0.4814	0.4834	0.4855	0.4876	0.4897	0.4918	0.4939	0.4960	0.4981	0.5002
.44	0.5023	0.5044	0.5066	0.5087	0.5108	0.5129	0.5151	0.5172	0.5194	0.5215
.45	0.5236	0.5258	0.5280	0.5301	0.5323	0.5344	0.5366	0.5388	0.5410	0.5431
.46	0.5453	0.5475	0.5497	0.5519	0.5541	0.5563	0.5585	0.5607	0.5629	0.5651
.47	0.5673	0.5696	0.5718	0.5740	0.5762	0.5785	0.5807	0.5830	0.5852	0.5875
.48	0.5897	0.5920	0.5942	0.5965	0.5987	0.6010	0.6033	0.6056	0.6078	0.6101
.49	0.6124	0.6147	0.6170	0.6193	0.6216	0.6239	0.6262	0.6285	0.6308	0.6331
.50	0.6354	0.6377	0.6401	0.6424	0.6447	0.6470	0.6494	0.6517	0.6541	0.6564
.51	0.6587	0.6611	0.6634	0.6658	0.6682	0.6705	0.6729	0.6753	0.6776	0.6800
.52	0.6824	0.6848	0.6872	0.6895	0.6919	0.6943	0.6967	0.6991	0.7015	0.7039
.53	0.7063	0.7087	0.7112	0.7136	0.7160	0.7184	0.7208	0.7233	0.7257	0.7281
.54	0.7306	0.7330	0.7355	0.7379	0.7404	0.7428	0.7453	0.7477	0.7502	0.7526
.55	0.7551	0.7576	0.7601	0.7625	0.7650	0.7675	0.7700	0.7725	0.7749	0.7774
.56	0.7799	0.7824	0.7849	0.7874	0.7899	0.7924	0.7949	0.7975	0.8000	0.8025
.57	0.8050	0.8075	0.8101	0.8126	0.8151	0.8177	0.8202	0.8227	0.8253	0.8278
.58	0.8304	0.8329	0.8355	0.8380	0.8406	0.8431	0.8457	0.8483	0.8508	0.8534
.59	0.8560	0.8586	0.8611	0.8637	0.8663	0.8689	0.8715	0.8741	0.8767	0.8793
.60	0.8819	0.8845	0.8871	0.8897	0.8923	0.8949	0.8975	0.9001	0.9027	0.9053
.61	0.9080	0.9106	0.9132	0.9158	0.9185	0.9211	0.9237	0.9264	0.9290	0.9317
.62	0.9343	0.9369	0.9396	0.9423	0.9449	0.9476	0.9502	0.9529	0.9555	0.9582
.63	0.9609	0.9635	0.9662	0.9689	0.9716	0.9742	0.9769	0.9796	0.9823	0.9850
.64	0.9877	0.9903	0.9930	0.9957	0.9984	1.0011	1.0038	1.0065	1.0092	1.0119

k^2	0	1	2	3	4	5	6	7	8	9
.65	1.0147	1.0174	1.0201	1.0228	1.0255	1.0282	1.0309	1.0337	1.0364	1.0391
.66	1.0418	1.0446	1.0473	1.0500	1.0528	1.0555	1.0583	1.0610	1.0637	1.0665
.67	1.0692	1.0720	1.0747	1.0775	1.0802	1.0830	1.0858	1.0885	1.0913	1.0940
.68	1.0968	1.0996	1.1023	1.1051	1 1079	1.1107	1.1134	1.1162	1.1190	1.1218
.69	1.1245	1.1273	1.1301	1.1329	1.1357	1.1385	1.1413	1.1441	1.1469	1.1497
.70	1.1524	1.1552	1.1580	1.1608	1.1637	1.1665	1.1693	1.1721	1.1749	1.1777
.71	1.1805	1.1833	1.1861	1.1889	1.1918	1.1946	1.1974	1.2002	1.2031	1.2059
.72	1.2087	1.2115	1.2144	1.2172	1.2200	1.2229	1.2257	1.2285	1.2314	1.2342
.73	1.2370	1.2399	1.2427	1.2456	1.2484	1.2512	1.2541	1.2569	1.2598	1.2626
.74	1.2655	1.2683	1.2712	1.2740	1.2769	1.2798	1.2826	1.2855	1.2883	1.2912
.75	1.2941	1.2969	1.2998	1.3026	1.3055	1.3084	1.3112	1.3141	1.3170	1.3198
.76	1.3227	1.3256	1.3285	1.3313	1.3342	1.3371	1.3400	1.3428	1.3457	1.3486
.77	1.3515	1.3544	1.3572	1.3601	1.3630	1.3659	1.3688	1.3716	1.3745	1.3774
.78	1.3803	1.3832	1.3861	1.3890	1.3918	1.3947	1.3976	1.4005	1.4034	1.4063
.79	1.4092	1.4121	1.4150	1.4179	1.4208	1.4237	1.4266	1.4294	1.4323	1.4352
.80	1.4381	1.4410	1.4439	1.4468	1.4497	1.4526	1.4555	1.4584	1.4613	1.4642
.81	1.4671	1.4700	1.4729	1.4758	1.4787	1.4816	1.4845	1.4874	1.4903	1.4932
.82	1.4961	1.4990	1.5019	1.5048	1.5077	1.5106	1.5135	1.5164	1.5193	1.5222
.83	1.5251	1.5280	1.5309	1.5338	1.5367	1.5396	1.5425	1.5454	1.5483	1.5512
.84	1.5541	1.5570	1.5599	1.5628	1.5657	1.5686	1.5715	1.5744	1.5773	1.5802
.85	1.5831	1.5860	1.5889	1.5918	1.5947	1.5976	1.6005	1.6034	1.6063	1.6092
.86	1.6121	1.6150	1.6178	1.6207	1.6236	1.6265	1.6294	1.6323	1.6352	1.6381
.87	1.6410	1.6438	1.6467	1.6496	1.6525	1.6554	1.6583	1.6611	1.6640	1.6669
.88	1.6698	1.6726	1.6755	1.6784	1.6813	1.6841	1.6870	1.6899	1.6928	1.6956
.89	1.6985	1.7014	1.7042	1.7071	1.7100	1.7128	1.7157	1.7185	1.7214	1.7242
.90	1.7271	1.7300	1.7328	1.7357	1.7385	1.7414	1.7442	1.7470	1.7499	1.7527
.91	1.7556	1.7584	1.7612	1.7641	1.7669	1.7697	1.7726	1.7754	1.7782	1.7810
.92	1.7839	1.7867	1.7895	1.7923	1.7951	1.7980	1.8008	1.8036	1.8064	1.8092
.93	1.8120	1.8148	1.8176	1.8204	1.8232	1.8260	1.8287	1.8315	1.8343	1.8371
.94	1.8399	1.8427	1.8454	1.8482	1.8510	1.8537	1.8565	1.8593	1.8620	1.8648
.95	1.8675	1.8703	1.8730	1.8757	1.8785	1.8812	1.8840	1.8867	1.8894	1.8921
.96	1.8949	1.8976	1.9003	1.9030	1.9057	1.9084	1.9111	1.9138	1.9165	1.9192
.97	1.9219	1.9245	1.9272	1.9299	1.9325	1.9352	1.9379	1.9405	1.9432	1.9458
.98	1.9484	1.9511	1.9537	1.9563	1.9589	1.9616	1.9642	1.9668	1.9694	1.9720
.99	1.9745	1.9771	1.9797	1.9823	1.9848	1.9874	1.9899	1.9924	1.9950	1.9975
1.00	2.0000									

MECHANICAL ANALOGS AID GRAPHICAL FLOOD ROUTING[a]

Discussion by C. O. Clark

C. O. CLARK,[1] A.M. ASCE.—The mechanical analogs of aid in graphical flood routing presented are of interest to all who must manage the natural waters of rivers and streams, either by scheduling and alloting the time of availability of beneficial use of that water which is harnessed and under control, or in warning of the unavoidable rampages of the public enemy when it is not under control.

The philosophy by which the author searches for his mechanical analogs, and finds the dampening effect of storage to be kin to the effect on the movement of a rear auto wheel of the distance between the wheels, makes refreshing reading. Such refreshment is needed by those who find the need for thinking and the opportunity for it completely eliminated by the codes, rules and regulations, and "accepted" formulas which make up much of the necessary pieces of routine engineering work. There are excellent examples of applied logic and scientific observation in the manner by which each of these analogs has been developed and built into a solid tool. There is food for thought in the fact that the author who conceived and built electrical analog computers for the solution of these problems, has now turned to mechanical analogs.

One of the analogs is as small and as subtle as a sliderule, and is as personally possessable as a straight edge/or a pencil. It is a truly Euclidian tool. Even more, by an as yet unproved modification of this tool, the author has opened a promising door toward a variable, non-constant, kind of solution of a problem which exists in the nature of streams as they are, but which solution has defied the limitations of brains, two-dimensional paper and the timeclock up to now, namely the class of problems involving variable relations between storage and discharge, variable lag and storage.

The writer is interested in the template type analogs, but this does not detract from the more mechanical ones, which are certainly ingenious, probably accurate, and based on sound analogy. This interest is very strongly influenced by the fact that the underlying principle of the templates, which Mr. Kohler now applies to long reaches of the main stem of a river system, is the same principle which underlies an application to an entire watershed to develop the type of flood hydrographs which the writer could call unit hydrographs, if they had a unit duration of rainfall, but must call instantaneous hydrographs when that unit is zero. This essential principle is that two steps of simple analysis are first found(1) to be essentially comparable to the more complex and simultaneous action of aquatic storage as it occurs in nature and as McCarthy approximated it in flood routing formulas developed for the Muskingum watersheds in Ohio.(2)

a. Proc. Paper 1585, April, 1958, by Max A. Kohler.
1. Hydr. Engr., Tulsa, Okla.

The author calls it lag routing, or lagging and then routing through storage, which he explains is bodily translation in time of a hydrograph without distortion, followed by a distortion or attenuation much in the manner effected by storage in a large level reservoir where storage is strictly a function of instantaneous outflow. This is a mental analog, of course, because the one kind of a waterway in which none of these mechanical analogs is needed is the level reservoir of great depth. Nevertheless, it is a correct analogy and it is the limiting case of a class of problems which vary from the headwater problems of sewer inlets where translatory effects have been considered large and ponding effects negligible, down through the streams which embody both in every proportion, onward to the large reservoirs, great lakes, and then to the deep seas in which the largest waves made by floods are but ripples on a surface translated in negligible time and at velocities compared with that of sound, and involving no appreciable lateral movement of mass. This idea is not new. Translation or lagging was built into flood calculation and thought in the so-called rational method of decades ago, repeated but not originated by Ramser in 1927. It was applied to urban problems of storm sewer design before the turn of the century, one writer says by Mulvaney in 1854, and to the rural problems of drainage at the time that Yarnell and Ramser were young men in these fields. It is just as natural for a headwater engineer to think solely in terms of translation, or a horizontal displacement in time and space, as it is for a downstream reservoir or maritime engineer to think of storage and of waves as something that involves only vertical displacement in space. Puls,(4) working on large tributaries of the Ohio River was one of the first to show that even the translation of river waves involved great storage, like reservoirs, but not so effective in attenuation. The writer has tried to knit together these upstream ideas of time required for concentration and the downstream ideas of storage which must be filled before flow can become steady, into the concept of an instantaneous hydrograph(1) which is the least size of hydrograph that could have a peak and the minimum duration if all elements of the watershed were equally involved. Hence it is dealing with a minimum volume associated with a flow and a duration and an influence of each on the other and of upstream and downstream elements on each and both, or the minimum wave which is stable enough in form to establish essentially steady flow and transmit it along a sloping channel.

What is definitely new in this work of Kohler's is the correlation of translatory storage with upstream storage, or lag, the downstream influences with change in shape as distinct from that of pure translation. It has been done before, and justified empirically or non-dimensionally in several trial-and-error or convenient solutions. But in words Kohler seems to be the first who has said lag is a function of inflow and attenuation is a function of outflow and who, in dimensions, has put one on one side of a line measured one direction and the other on the other side of the line measured the other direction — as he has in his template of figure 7.

The writer was not prepared to accept this at face value as rational. But being already committed to an empirical usage of that principle, and having the means of experimentation at his disposal, he caused two flood waves to be generated by purely upstream forces, within the banks of a known stream, and without the lateral inflow which confuses natural flood records. Nor was there the lack of knowledge of the exact form of the upstream wave as is the case with most rain-generated waves. The stream is steep enough so that stage-discharge relations are as constant as they can be in ordinary

streamflow hydraulics, and the friction factors are so much larger than momentum forces and velocity heads that influence of these things on stages and discharge is probably much less than it is in the streams for which flood routing techniques would be applied. The results are summarized here as support for the author's view (that where the Muskingum routing equations apply, i.e. constant time elements and linear storage-discharge relations, the effects of flood routing can be suitably approximated by transposition as a function of inflow and storage attenuation as a function of outflow). (The stream does not have the form or dimensions which would lead to intensification of wave form, so these forms are neither confirmed nor denied by this experiment.)

Two premises are common to the selection of the experimental wave forms, the author's lag routing templates, and the writer's concept of hydrograph analysis and synthesis by basin shaped instantaneous unit-graphs. These two premises are that, in a stream where time elements of flow are approximately equal regardless of flow magnitude,

1) Storage effects may be approximated by two kinds of storage in suitable combination which tend to attenuate flow and which do not, and,

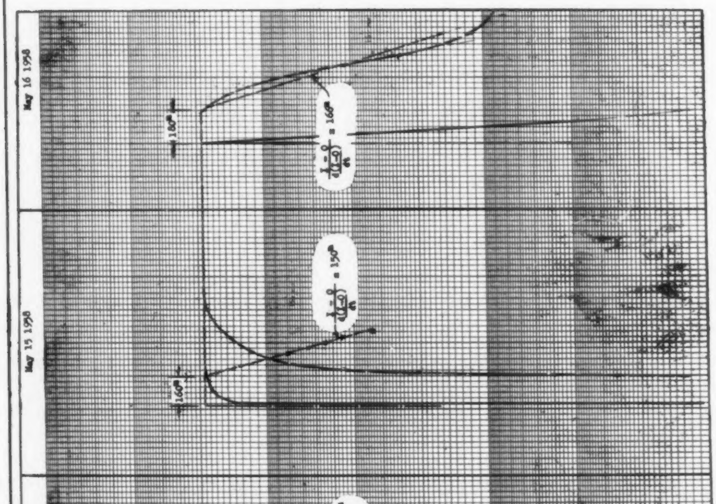
2) Duration of flow and increasing and decreasing flows may be approximated by small instantaneous impulses of positive or negative change, and any hydrograph no matter how complex can be approximated by some sequence of impulses.

In the writer's view, premise (1) is thought of first as amounts of storage which translates and attenuates, or reductive storage, and some which translates and accentuates peaks, with the latter apparently a function of inflow or upstream factors and the former a function of outflow or downstream factors. This theory is the wedge-storage concept familiar in the Burns-Harkness-McCarthy equations. However, these wedges, or two components, can likewise be combined in combinations of two prisms so that tendancies to build up and to flatten are balanced and only translation results, and another component providing the net attenuation which is common in most streams (or the build up which is possible in some cases and would apparently involve negative values of the K-item). This prism combination is Mr. Kohler's division into lag-and-storage values. (The writer would caution that both items represent storage, however, and that translatory storage and reductive storage are suitable names which retain the concept of physical storage space required in a channel to produce the effects involved).

With the above premises in mind, the experiments here reported were designed to release two rectangular wave forms of different magnitude by suddenly opening and suddenly closing gates with suitably sustained periods between each change to permit flow to become completely stable and steady between changes and steady throughout the zone of the experiment.

The resulting wave was observed as river stage at two points, 0.3 and 6.7 miles downstream and at five-minute intervals. Using the two stages of positive steady flow observed and the initial stage as zero flow, an approximate rating curve was prepared for each observation point and the stage observation converted to approximate steady flow in much the manner which is conventional in stream gaging on the steep streams to which these premises can apply.

The resulting interpretation is shown on figure 1 hereof. The lag and



FLOW FROM QUICKLY OPENED AND CLOSED GATES AND AT POINTS 0.3 AND 6.7 MILES DOWNSTREAM

COMPARED WITH LAG AND ROUTING VALUES WHICH NEARLY EQUAL THEM

reductive storage values are dimensioned only for the downstream point. The K values for the descending hydrographs, defined as

$$\frac{I - 0}{\frac{d(I-0)}{dt}}$$

in the general case, or when inflow has ceased as

$$\frac{(0)}{\frac{d0}{dt}}$$

are the conventional ones. In this case, unlike natural hydrographs in which inflow is never steady except at zero, a determination is also possible on the ascending leg, and the values are shown.

Within the range of many practical uses, the values rising and falling and the associated lags are constant. Whether the small differences in the time elements as printed on the figure are factual or the faults of methods of observation or deduction is not known, and the opportunity to repeat the experiments suitably has not since presented itself.

The measure of the whole storage utilized in wave passage is the sum of the associated lag and recessional values, successively 258, 298, 310 and 346 minutes. As volumes these are respectively 0.36, 0.41, 0.43, and 0.48 acre-feet per cfs of flow, or nearly 800 acre-feet in the prism of changing water level in the range of these observations. The writer wishes to emphasize that these are physical volumes, and changes in channel alignment or confinement could alter them in proportion to those changes.

This is a stream where the proportion of storage along the channel to flow in the channel is about the same throughout the range of in-channel flow rates, as has been confirmed in backwater computations using channel cross-sections. Hence, this is a stream to which constant time elements apply essentially within the channel and the range of these observations. The writer cautions against too rapid extension of this experience into streams where constant time elements do not apply, and where linear storage-discharge relations are not a fact. All of the "black marks" on the record of the McCarthy flood routing method and on unit hydrograph theory have been put there by people who did not first find these constant time elements or linear relationships to be facts as applicable in their subject streams. Where these relationships are factual, the theories work very well. However, where the applicable facts are that the storage-discharge curves are wide and complex loops, and there is no fixed relationship between stage and flow rate in the first place, more basic and more fundamental methods must be used. The variable lag-variable reductive storage templates illustrated by Mr. Kohler offer more promise than anything else yet available, but, like many other loop relationships, may not be constant from flood to flood and wholly precise except in the flood from which the loop or template is derived. Nevertheless, since the streams to which such loops apply have other severe complications of measuring discharge and of shifting relationships, this is only a small hazard in the total problem.

On the positive side from this cautionary expression, the variable templates are capable of indicating the earlier arrival of very large flood peaks and the almost complete flattening of small floods just beyond the stages of bank overflow, which phenomena are the characteristics of many larger streams and

probably of all aggrading, silt-carrying streams (like the Verdigris River which Mr. Kohler used as an example for his template of figure 7 and illustration of the presented idea). Because the template is adapted to the simplest of graphical methods of solution, and involves no reading, tabulation, replotting or reprinting, but produces a hydrograph from a hydrograph, it is the most promising presently available answer to some difficult flood-time problems. One of these is the solution for many streams almost simultaneously, as in the emergency of flood-time, and where constantly changing data are being received and there is constant demand for the changes in predictions or operations expected from the last few hours of rainfall, the breach of some levee, or the change in gate setting at some flood control or power dam. Under these conditions the efforts of several men must be quickly applied under emergency conditions and can be safely done with the templates carefully prepared under the more leisurely conditions of other times.

The template is the newest and most unique of recent flood tools. It is destined to rank for several decades with Sherman's unithydrograph as a basic simplification of essential facts for practical daily work.

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3. Runoff from Small Agricultural Areas, by C. E. Ramser, Journal of Agricultural Research, Vol. 34, No. 9, May 1, 1957.
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NORTHEASTERN FLOODS OF 1955: FLOOD CONTROL HYDROLOGY[a]

Discussion by Leo R. Beard

LEO R. BEARD.—This paper by Mr. Childs admirably documents the hydrologic aspects of a record-breaking flood event and their effects on the design of flood-control structures. The writer is particularly interested in the author's discussion relative to flood frequencies. The author justifiably feels that "flood frequencies for stations with 100 years of record will give fairly reliable frequencies up to 50 years. Beyond this point the reliability drops rapidly and beyond 100 years the frequency is fictitious." However, estimates of extreme flood frequencies or probabilities are necessary to an adequate economic evaluation of project justification, and proper planning therefore requires that satisfactory methods of extrapolating frequency curves be developed if at all feasible. The writer has briefly investigated this problem along lines explored by Messrs. Russell Morgan, Marshall Iakisch and others of the Philadelphia District, Corps of Engineers, in their studies of the Delaware River Basin. It is hoped that the following brief description of this investigation will shed some light on the evaluation of future frequencies of extreme hurricane floods such as those of August 1955.

Experience in computing flood frequencies for the New England area has been particularly discouraging. While in most of the United States annual maximum flood events are apparently distributed in reasonable accord with the normal probability curve, there has been strong evidence that the New England area experiences an unusually large number of major floods (that is, that the frequency curves are skewed in the positive direction). Until the 1955 floods occurred, there was some debate as to whether the observed excess of large floods has been accidental (due to chance) or whether it derives from some physical characteristic peculiar to the New England area. With the added evidence from the 1955 floods, little doubt remains that some factor is operating in New England floods that is not characteristic of floods in the rest of the nation.

The author suggests that "large floods and the smaller events are not related and should not be compiled in the same array and analyzed by statistical methods," and "analysis of floods should be divided into two categories: one for moderate annual floods, and the second for the larger floods." The writer believes, however, that those factors causing a major event can also combine such as to cause a small event, and consequently that some small events must be related to the large floods. It is doubtful that there is an abrupt discontinuity in channel conveyance factors such that they are distinctly different for "large" and "small" floods.

However, it is agreed that events which are not related (that is, not caused by the same general factors) should not be combined in a single frequency array. Since most of the New England floods are caused by extratropical

a. Proc. Paper 1663, June, 1958, by Elliot F. Childs.

cyclones but a few are caused by tropical cyclones (hurricanes), it has been suggested that events be segregated into two groups on this basis. In this way, perhaps theoretical distribution functions used successfully in other regions might be applied in the New England area. Such functions are requisite to accurate evaluation of extreme flood frequencies.

This idea which was developed in connection with Delaware River studies, was tested by the writer in a study conducted under one of the Civil Works Investigations Projects of the Corps of Engineers. Twelve long-record stream gaging stations distributed throughout New England were selected for the study. The period of record considered was limited to the last 40 years, because data on hurricanes during earlier years are not reliable. Maximum mean-daily flows were selected in three series as follows:

a. Annual maximum non-hurricane floods
b. Annual maximum hurricane floods
c. Annual maximum floods, regardless of type

The list of hurricane floods used was compiled from various hurricane reports. The various reports available were not consistent, and it is felt that, particularly in the early years, some of the actual hurricane events have not been listed. Hurricane floods used in the study are as follows:

Hurricane Flood Dates by 10-year Periods

1916-25	1926-35	1936-45	1946-55
23 Jul 1916	5 Oct 1927	19 Sep 1936	30 Aug 1949
1 Oct 1920	11 Aug 1928	21 Sep 1938	20 Aug 1950
25 Oct 1923	3 Oct 1929	22 Aug 1939	12 Sep 1950
27 Aug 1924	17 Sep 1932	2 Oct 1939	2 Sep 1952
1 Oct 1924	24 Aug 1933	2 Sep 1940	15 Aug 1953
	18 Sep 1933	19 Sep 1940	8 Sep 1953
	20 Jun 1934	17 Oct 1943	31 Aug 1954
	9 Sep 1934	2 Aug 1944	12 Sep 1954
	5 Sep 1935	15 Sep 1944	16 Oct 1954
		22 Oct 1944	14 Aug 1955
		27 Jun 1945	18 Aug 1955
		19 Sep 1945	21 Sep 1955

Since the frequency curve of hurricane floods is based on fewer events than the years of record, plotting of the curve was based on the percentage of floods rather than on the percentage of years. In order to convert the frequency in percentage of floods to the frequency per hundred years, the indicated exceedence frequency must be multiplied by the ratio of the number of floods used to the number of years of record. This adjustment of the hurricane frequency curves is illustrated in Figure 1.

Frequency curves for one of the stations studied are shown in Figure 1. It was found that, in the case of hurricane floods, the plotted data fit the computed log-normal curves reasonably well in most cases, but occasionally suggest an upward curvature of the frequency curve. The plotted data of the non-hurricane floods almost consistently show an upward curvature of the frequency data relative to the log-normal distribution.

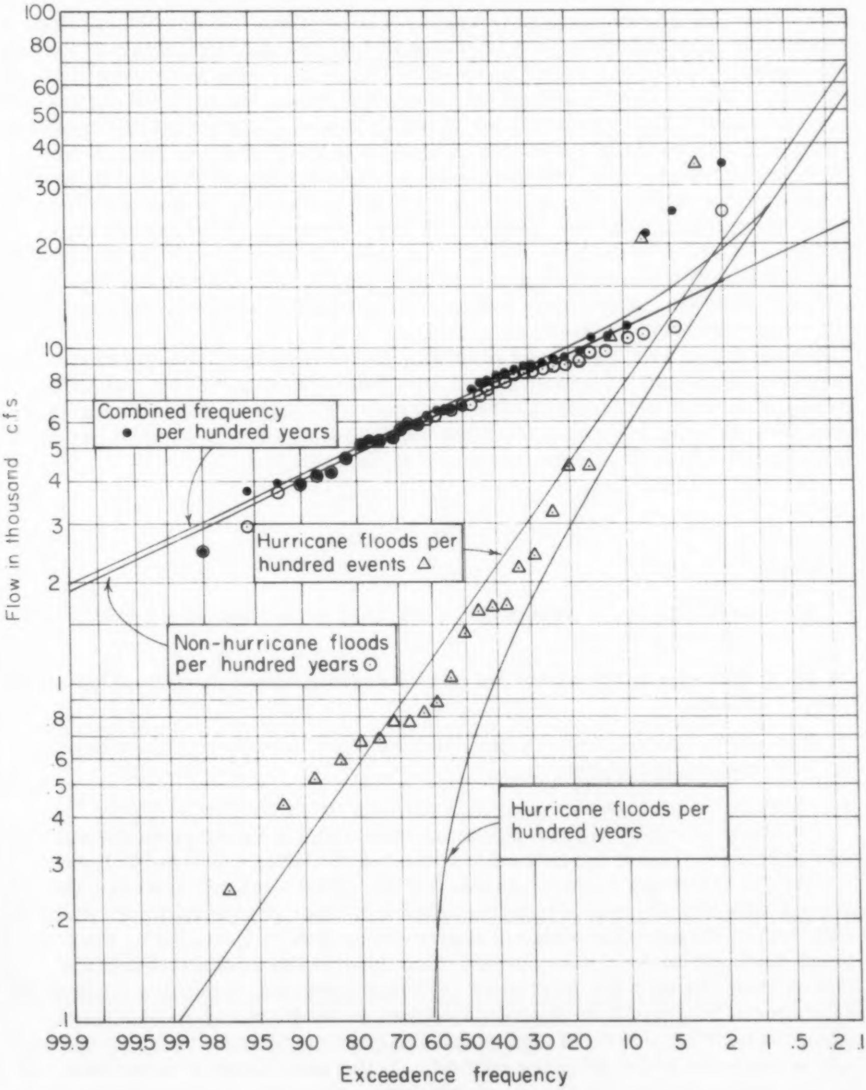

Figure 1

COMPARISON OF COMPUTED AND OBSERVED FLOOD FREQUENCIES
QUINEBAUG R. AT JEWETT CITY, CONN.

In order to summarize the data for all twelve stations in such a way as to demonstrate the average curvature, the floods at all stations that were in excess of the indicated 100-year value were counted. Similarly, the floods that were between the 50-year and 100-year indicated values were counted, and so forth until the entire range of frequency was covered. Comparison of these aggregate numbers with those indicated by the straight line frequency curve is shown in Figure 2. It will be noted in Figure 2 that the hurricane-type floods evidence a slight upward curvature throughout the range of the frequency curve. The non-hurricane floods evidence a sharp upward curvature only in the upper range of floods. Since it was known that the largest non-hurricane general flood that has occurred in New England in the past 300 years is that of 1936, it was felt that perhaps this one flood may have been unduly responsible for the upward curvature shown by the non-hurricane floods. In order to examine the effect of this one flood, the 1935 flood events were subtracted from the frequency count and the aggregate points replotted. This comparison, also shown in Figure 2, is somewhat better, but there still is a tendency toward upward curvature.

When floods are segregated according to cause or type, and where the separate frequency curves of annual maximum events are plotted as in this study, it is often desirable to obtain a frequency curve of annual floods regardless of type. In order to do this, the following formula derived in the Philadelphia District, Corps of Engineers, is employed:

$$\frac{P_3}{100} = \frac{P_1}{100} + \frac{P_2}{100} - \frac{P_1 P_2}{10{,}000}$$

in which:

P_1 = the exceedence frequency per hundred years, annual maximum non-hurricane floods

P_2 = exceedence frequency per hundred years, annual maximum hurricane floods

P_3 = exceedence frequency per hundred years, annual maximum floods, regardless of type

Combination of frequencies using this formula is illustrated in Figure 1.

As might be expected from the experience with the separate hurricane and non-hurricane flood frequency curves described above, combination frequency curves computed in the above manner for the 12 stations revealed that the observed annual maximum events (regardless of type) at practically all stations are more frequent in the range of higher floods than is indicated by the computed combination frequency curves. This may be due to the fact that the March 1936 and even the November 1927 non-hurricane floods are much more rare events than would be expected to occur in the 40-year period studied. Also, the hurricane flood of August 1955 is considered to be much rarer than can be expected in the 40 years studied. On the other hand, it is possible that the log-normal technique employed, although successful in most regions of the United States, is not applicable in New England. Consideration was given to eliminating this disparity by flood segregation other than between hurricane and non-hurricane types, but it was concluded that other flood types were not distinct or sufficiently independent.

If the results of the study described herein were the only evidence available

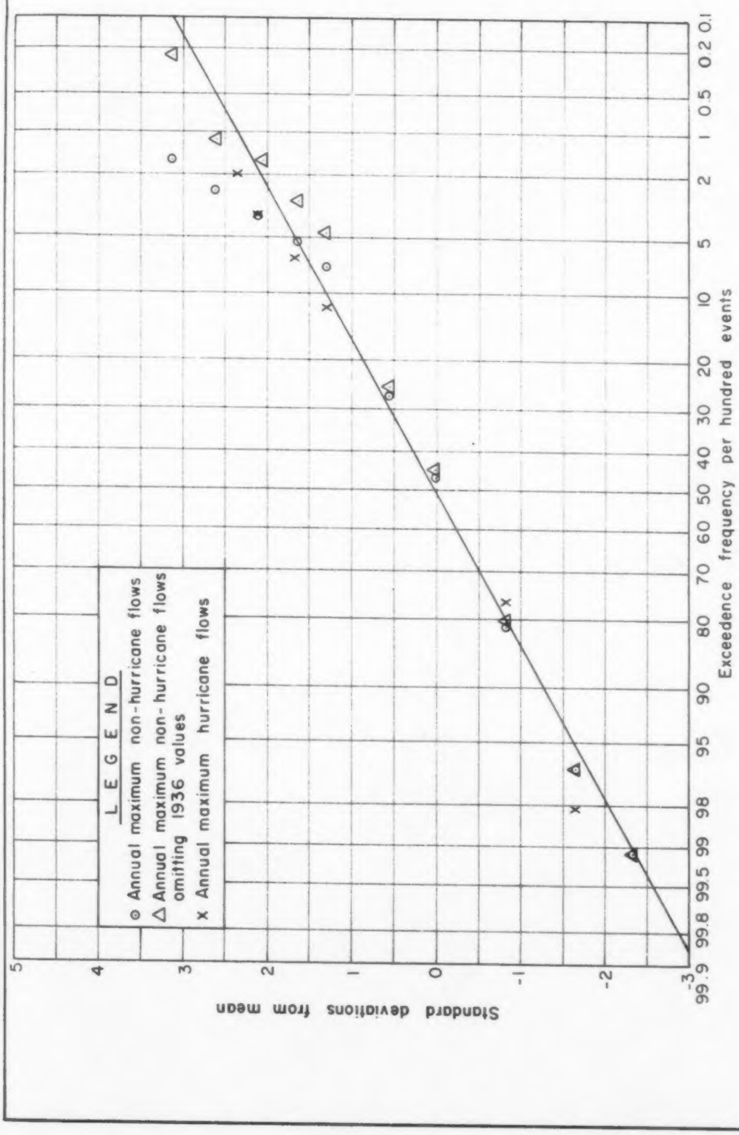

Figure 2

COMPARISON OF TOTAL FREQUENCY COUNT FOR 12 STATIONS

for judging the adequacy of the techniques employed, the excess of experienced large floods would cast strong doubt on their validity. However, the log-normal technique has been successfully employed in most other sections of the country, and it is only reasonable to expect a serious excess of large floods in some regions and a serious shortage of large floods in other regions. As a matter of fact, there is a feeling among hydrologists familiar with the New England region that the floods of 1936, 1955, and even 1927 are indeed very rare events, particularly in the light of historical data. It is believed, therefore, that the application of the log-normal technique to the frequency analysis of hurricane floods and non-hurricane floods separately constitutes a reasonable method for evaluating the future frequency of extreme floods in the New England area. The frequency curves derived by this method indicate that the 1955 floods were moderate at 9 of the 12 stations studied, but that the flood of 18 August (Hurricane Diane) will be exceeded only once in 200 to 300 years at the remaining three stations.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502(IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(ST1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520(EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568(WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588(HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616(HY2)[c], 1617(SU1)[c], 1618(PO2)[c], 1619(EM2)[c], 1620(CP1)[c].

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629(ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(WW3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647(SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666(PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675(PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684(HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710(ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719(EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733(PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742(PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775(IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784(IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806(HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815(ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824(SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HY5)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HW3)[c].

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850(SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858(SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(IR4), 1864(IR4), 1865(ST8), 1866(ST8), 1867(ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876(WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(IR4)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(ST8), 1891(PP1)[c].

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JANUARY: 1892(AT1), 1893(AT1), 1894(AT1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(ST1)[c], 1917(AT1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1)[c], 1924(HY1)[c], 1925(HY1)[c], 1926(PL1), 1927(HW1), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

(c) Discussion of several papers, grouped by divisions.

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